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THE UNIVERSITY OF ALBERTA

A STUDY OF DELAYED FAILURE OF A CUT SLOPE IN STIFF CLAY

by

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A THESIS
SUBMITTED TO THE FACULTY OF GRADUATE STUDIES AND RESEARCH
IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE
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THE UNIVERSITY OF ALBERTA

FACULTY OF GRADUATE STUDIES AND RESEARCH

The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies and Research, for acceptance, a thesis entitled ..A Study of Delayed......
Failure of a Cut Slope in Stiff Clay..............................
....
submitted by ..Bruce H. Kjartanson........................
in partial fulfilment of the requirements for the degree of Master of Science in Civil Engineering.

S Thomson
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Supervisor
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Date..May 30, 1978..............

ABSTRACT

A number of slope failures occurred along a 400 metre section of a freeway cut in southwestern Edmonton after the completion of construction in 1969. The cut in this section, which is approximately 7.3 metres deep on a slope slightly greater than two to one, was excavated entirely in the lightly overconsolidated, fissured and jointed Lake Edmonton sediments. The pre-construction topography was essentially flat and evidence of in-situ pre-shearing has not been found; thus, the failures are considered to be time-delayed, first-time slides.

A comprehensive laboratory testing program, involving index tests, consolidation tests and triaxial and direct shear tests allowed an assessment of the influence exerted by such factors as structural discontinuities and compositional heterogeneity on the strength and stress-strain/volume change characteristics of the geologically complex Lake Edmonton sediments. One class of materials tested, termed the "Weathered Material" (from the upper few metres), displayed a bi-modal failure characteristic which was intrinsically related to a nugget or blocky structure. Above the in-situ overburden pressure, failure occurred through the intact lumps resulting in peaked sress-strain curves and subdued dilation after the point of peak deviator stress. Below the overburden stress, however, failure

occurred along the nugget discontinuities resulting in a flatter stress-strain curve and pronounced dilation after the peak deviator stress. The material behaved as a cohesionless, granular aggregate below the overburden stress. Material sampled from a depth of about five metres below the crest of the slope was composed predominantly of a medium plastic silt and a high plastic, fissured clay in varying proportions. The strength of this material was found to depend on the location and orientation of the weaker component in the system, that is, the fissured clay.

The processes of physical weathering, such as cycles of wet-dry and freeze-thaw, can destroy an intact soil structure in a few seasons in a severe climate such as that of western Canada. The weathering processes appear to increase the srength of the intact lumps or nuggets between the fractures through a net consolidation or densification effect, but to decrease the mass strength of the material under low normal stresses, reducing it essentially to a cohesionless, granular aggregate.

The results of a stability analysis conducted on one of the landslides, which occurred in August, 1973, showed that the stability was dependent on the strength associated with the fissured, high plastic clay as well as partial softening in the central portion of the slope, on the destruction of the effective cohesion due to weathering in the surficial zones and on a buildup of hydrostatic pressure in the structural discontinuities due to infiltration of rainfall and surface runoff. Moreover, other slope failures on the site have been related to intense periods of rainfall. It should also be noted that it is of utmost importance to conduct laboratory strength tests under the stress ranges encountered in the field.

Research into the origin and factors controlling the distribution of fissures and joints in soil masses should be undertaken. In addition, creep or relaxation tests should be conducted to evaluate the time-dependent deformation properties of the Lake Edmonton Sediments and to check if the short-term laboratory peak strength may actually be bypassed in the long-term condition.

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CHAPTER I

INTRODUCTION

1.1 The Whitemud Freeway Cut Failures

During the late 1950's and early 1960's, extensive urban development necessitated the construction of a major freeway system in the southern and south-western sections of Edmonton (Figure 1.1). To reduce the approach fill on the South bank of the North Saskatchewan River and to avoid a large cut in the north bank, the route centerline was located in the Quesnell Ravine. A suitable grade was established up the ravine with a minimum of cut and fill; however, west of 149th Street, where the right-of-way left the ravine, a cut was necessary to maintain the grade.

Due to the proximity of residential development, the width of the right-of-way was limited; therefore, the side slopes were excavated slightly steeper than two horizontal to one vertical (Soderberg, 1978). A number of slope failures occurred along a 400 metre section of the cut between 156th and 159th Streets after the completion of construction in 1969 (Figure 2.3). The cut in this section, which is approximately 7.3 m. deep, was excavated entirely in the overconsolidated Lake Edmonton Sediments (geology discussed in the following chapter).

The following methods have been utilized to stabilize the slopes:

1. Slope flattening (slide #1, Figure 2.3).
2. Complete excavation of the failed mass and rebuilding the slope with a coarse, pit-run granular material (slide #6).
3. Benching, with the installation of gravel drainage mats on the benches (slides #4, #5, and #7).
4. The installation of a concrete, crib-type retaining wall (slides #2 and #3).
5. The installation of counterfort drains with cast-in-place stitch piles (slide #8).

The Whitemud Freeway cut failures thus offered an interesting and complex slope stability problem. The pre-construction topography was essentially flat (Chapter II), thus the failures were first-time slides in an overconsolidated material (evidence of pre-shearing has not been found). The slides have failed along non-circular slip surfaces, which may indicate bedding plane or some other type of control, such as the presence of the underlying till.

This thesis concentrates on the slide which occurred in August of 1973 (the 1973 slide ; Plate 1), because much survey and sub-surface data was available from an extensive site investigation undertaken in the fall of that year. Undisturbed soil samples were collected from the vicinity of the 1976 slide (Plate 2) in the spring of 1977.

1.2 Time-Dependent, First-Time Slides in Discontinuous, Overconsolidated Clays.

The term "time-dependent, first-time slide" is used here to indicate failure in a previously unsheared material which occurs some time after a slope has been cut or was eroded; i.e. the slip does not involve reactivation of an old slide mass or movement along a pre-existing shear surface. Discontinuous, overconsolidated clays have been troublesome, as natural slopes and excavations often fail years later at strengths much below the laboratory peak strengths (James, 1970). There are, therefore, time-dependent processes at work which are causing a significant reduction in the operative strength of these materials and which subsequently lead to long-term slope instability.

Morgenstern (1977) has distinguished two such processes which may lead to strength deterioration of stiff fissured, dicontinuous clays with time; viz, delayed and progressive failure. Morgenstern stated:

> "...Delayed failure is taken here to include all processes that contribute to the reduction of shear strengths with time".

One such process is pore pressure equalization, which essentially involves the dissipation of negative excess pore pressures (generated by the stress relief of excavation) to the long-term or steady-state condition. Another common process is softening, which effectively reduces the shear strength from the peak to the fully softened strength (i.e.

$c'=0$, $\phi'=\phi$peak).

The other process, referred to as progressive failure involves:

> "...The non-uniform mobilization of shear strength along a potential slip surface. If the soil has work-softening stress-strain characteristics and it is being deformed to failure, some elements will reach peak strength before others and therefore when failure finally ensues, some elements will be beyond peak strength while others have just reached it."

Analyses would indicate the mobilization of an average shear strength less than the peak, i.e. $c'=0$ with ϕ' less than the peak value; the limiting ϕ' being the residual value.

A number of theories and mathematical models have been postulated to account for the mechanism of progressive failure. Skempton (1964), when formally introducing the concept of residual strength, proposed a theory of progressive failure in which structural discontinuities act as stress concentrators:

> "...Not only will the fissures and joints reduce the average strength of the clay mass, but they can cause the peak to be crossed, as a result of local overstressing and a progressive decrease in strength will follow."

His theory, however, is based on the premise that the strength would inevitably reach the residual condition everywhere along the incipient slip surface. Bishop (1967) suggested a mechanism based on local overstressing in terms of the shear stress (in the short-term, undrained condition) or the ratio of the shear stress to the effective normal stress (in the long-term, drained condition) in the slope; when the limiting value defined by an elastic solution is reached, local failure will occur. Bishop went on to explain that only changes in the loading conditions or pore pressures will lead to a progressive extension of the zone of failure along the potential slip surface, while within this zone the shearing resistance will begin to drop from the peak to the residual value.

Theories based on the release of high horizontal stresses upon excavation in overconsolidated materials have been proposed. Duncan and Dunlop (1969) with the use of a finite element stress analysis, have predicted the occurrence of localized failure in zones of overstress brought about by the release of high horizontal stresses. They have shown the effect increased markedly with higher values of Ko , while the variation in the modulus of elasticity had little effect. Yudhbir (1969) has also postulated a progressive failure mechanism based on the effects of the release of high horizontal stresses. On the basis of Ko -unloading oedometer tests, which resulted in shear failure of the specimens, he concluded that Ko - unloading in the field would cause failure down to a depth of forty feet. Additional shearing deformations associated with further unloading were believed to result in the propagation of the failure surface.

Bjerrum (1967) explained in great detail a progressive failure mechanism based on the release of the recoverable strain energy due to the effects of weathering on diagenetic bonds. He postulated that three conditions must be satisfied before the mechanism is possible:

1. Shear stress concentrations at the toe of the slope must be larger than the peak strength of the material.
2. The clay must contain a sufficient amount of recoverable strain energy.
3. The material must show marked work-softening characteristics.

Analytical studies have been undertaken to model the mechanism of progressive failure. For example, Christian and Whitman (1969) have theoretically illustrated the effects of horizontal stress release and non-uniform swelling in terms of elastic, plastic and strain-softening behavior. Their model was in terms of the bonding between a single layer of the material and a rigid base.

Lo and Lee (1973) have established a finite element solution of the stress distribution in a slope of strain-softening material and have shown that if the rate of decrease of the drained strength is known, the time of failure of excavations may be predicted. These analyses, however, are based on simplified stress-strain curves and very limited zones of failure. Moreover, they assume that all the elements in the slope follow a unique strain-softening characteristic, which in reality is not the case (Simmons, personal communication).

The preceding proposed mechanisms and analytical models of progressive failure have been well established on a hypothetical basis, but as Morgenstern (1977) pointed out:

> "...There appear to be no well-documented case histories of first-time slides in heavily overconsolidated soils to indicate that progressive failure plays a dominant role in governing stability."

For example, the Seattle Freeway slides in 1962 and 1963 which Bjerrum cited as evidence of his mechanism, have since been proven by Palladino and Peck (1972) to have failed along zones pre-sheared by periglacial processes. Furthermore, steep, high slopes which would certainly have non-uniform stress conditions, have remained stable for extended periods of time (e.g. Esu (1967) discussed such slopes in Italian clays). Several failures in intact overconsolidated clays, mobilizing both peak values of c' and ø' have also negated a progressive failure mechanism . For example, the Selset case (Skempton and Brown, 1961) involved the failure of a slope in an intact, heavily overconsolidated boulder clay which demonstrated perceptible strain-softening behavior; analyses conducted using a c' or ø' lower than the peak values resulted in unacceptably low factors of safety.

A case history, however, has recently been published by Burland et al (1977) which is in "broad agreement" with the mechanism proposed by Bjerrum (1967). Slip movements occurred in a 29 m. deep brickpit which was being excavated in the overconsolidated Oxford clay. The authors concluded:

> "Overthrusting relative to lower beds occurred by sliding on a shear band which developed by progressive failure as the maximum shear stress exceeded the peak shear strength along bedding planes near pit base level."

James (1970) analyzed more than fifty first-time slides in various overconsolidated clays of the United Kingdom in an effort to establish the validity of the progressive failure hypotheses. It was found that the majority of slides had failed with a value of effective cohesion approaching zero while maintaining an effective angle of shearing resistance at or greater than the laboratory peak value; hence, progressive failure was an uncommon occurrence. As a rational explanation as to why progressive failure is rare, James (1970) pointed out that large displacements, often in the order of feet, are necessary to develop residual conditions on a continuous slip surface. He quantified this in terms of the field strain, defined as the ratio of the amount of slip movement occurring to the length of the slip plane (approximately the ratio of the scarp height to the height of the slip). The results of the reduction in ϕ' calculated from slides in the London, Oxford and Lias clays plotted against field strain are shown in Figure 1.2.

Coupling these results with the work of Bishop and Lovenbury (1969), which essentially proved that there is no path to the residual which by-passes the peak, James (1970) came to the conclusion that :

> "...The need for relatively great deformations to produce residual conditions may now be considered again in relation to the hypothesis of Bjerrum (1967), or the hypothesis of lateral earth pressures, Yudbhir (1969). It is suggested that unless deformation is very much localized, along one thin layer or at the interface between two somewhat different layers the strains would be too small to give anything approaching the residual."

He went on to state that non-uniform swelling could also produce localization of strains, with residual conditions being attained at much lower deformations (analogous to those produced at the boundary of competent and incompetent beds during folding); thus the progressive failure mechanism would be made time-dependent.

In the light of the preceding work by James (1970) and the work of Skemptom (1966) and Morgenstern and Tchalenko (1967), which described the formation of a continuous shear zone from the progressive coalescence of minor (Riedel) shears, Skempton (1970) discussed two successive post-peak stages of development of first-time slides:

> "...a) dilatancy and the opening of fissures leading to increases in water content and culminating in a drop in strength to the fully softened value, at which stage there is a softened shear zone with numerous discontinuous shears;
> b) development of principal shears of appreciable length some of which eventually

> link together and form a continuous shear, when the residual strength is reached along the entire slip surface."

From the preceding discussions it therefore appears that time-dependent, first-time slides in discontinuous, overconsolidated clays may best be explained in terms of delayed failure, involving such processes as pore pressure equalization and softening. As Morgenstern (1977) has indicated, great care should be taken in the evaluation of post-construction pore pressures in order to distinguish between the alternative explanations of delayed failure. For example, Vaughn and Walbancke (1973) observed substantial negative pore pressures in a cut slope in London clay nine years after excavation. In re-analysing many slope failures in the London clay that had been mainly attributed to softening processes (e.g. James, 1970), they came to the conclusion that the slips were delayed primarily by pore pressure equalization. The process of pore pressure equalization is discussed further in Chapter IV. Various mechanisms of softening, which have been proposed through the years, are outlined in the following paragraphs.

A softening mechanism was first postulated by Terzaghi in 1936 to account for the time-dependent strength decrease and failure often observed in fissured, overconsolidated clays. This softening mechanism, subsequently applied to the analysis of failures in the London clay by Skempton (1948), involved the opening of fissures and joints due to the lateral stress relief upon excavation. Owing to the high strength of the clay, the discontinuities would remain open to great depth, hence, allowing ingress of water. The clay exposed on the fissure surfaces would begin softening, the material would deform and a stress redistribution would occur. The end product is a clay essentially reduced to its normally consolidated or "fully softened" state (Skempton, 1948).

Eigenbrod (1972) has pointed out that this type of softening does not depend on large deformations before failure, but is dependent on the presence of discontinuities. Furthermore:

> "...It is likely to be rather slow in general and noticeable strength decrease can be realized only after decades."

Skempton (1970) proposed a softening mechanism which was largely dependent on localized Riedel thrust shearing with subsequent softening in the post-peak stages, as has already been described. The displacement necessary to reduce an overconsolidated clay to the fully softened condition is several times greater than the displacement to the peak, but it is considerably less than that corresponding to the residual; i.e. displacements measured in inches rather than feet. Therefore, the displacements which result from unloading (overstressing) or the K_o -effect may be great enough to initiate the mechanism proposed by Skempton.

Morgenstern (1977) upon reviewing the studies of Patton (1966) and Marsland (1972) which suggest different modes of failure of discontinuous materials above and below a certain threshold normal stress, has postulated;

> "...The result of softening may simply be to reduce the dilatant characteristics of the fissured system at low effective stresses."

In other words, the geometric component of shear resistance (commonly referred to as the "i" angle) will be lost due to softening of the intact material between the fissures.

Another process of delayed failure, physical weathering, is discussed in greater detail in subsequent chapters.

1.3 Purpose of the Investigation

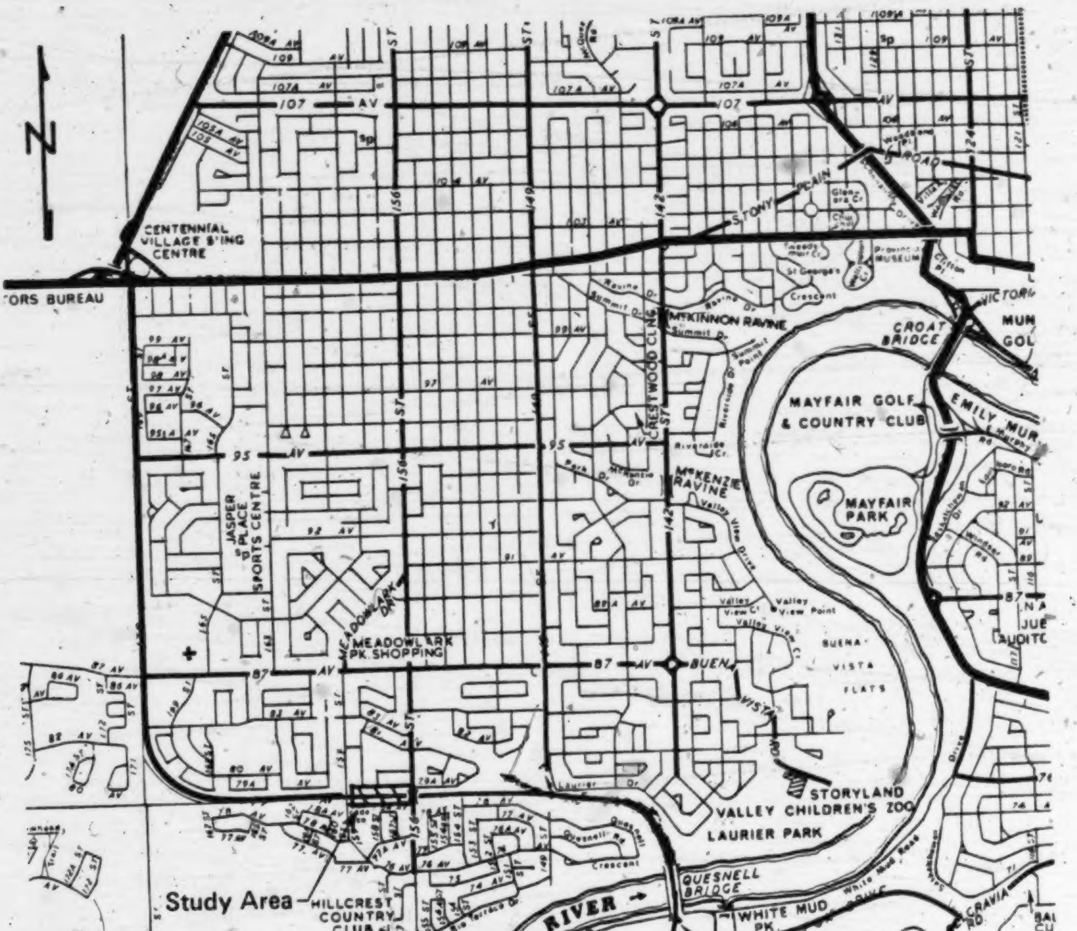
In the light of the preceding discussions on the nature of the Whitemud Freeway slides and progressive versus delayed failure mechanisms, the purpose of this thesis investigation is therefore :

1. To study and assess the factors, such as structural discontinuities and compositional heterogeneity which control the strength and stress-strain characteristics of the Lake Edmonton deposit.
2. To evaluate the effects of physical weathering processes such as cycles of wet-dry and freeze-thaw

on the strength of the soil, and to consider the time scale in which their effects become important.

3. To investigate the time-dependent aspect of the landslides and to propose a mechanism of failure, employing a stability analysis conducted on the 1973 slide with the strength parameters determined in the laboratory testing program.

4. To assess the effects of prolonged, heavy periods of rainfall on the stability of the slopes.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WHITE MUD CREEK
RAINBOW VALLEY CAMP GROUND
EDMONTON COUNTRY CLUB
COUNTRY CLUB ROAD
TERWILLIGAR PARK
DERRICK GOLF & WINTER CLUB
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Scale (km)

Figure 1.1 General Map: Site Location

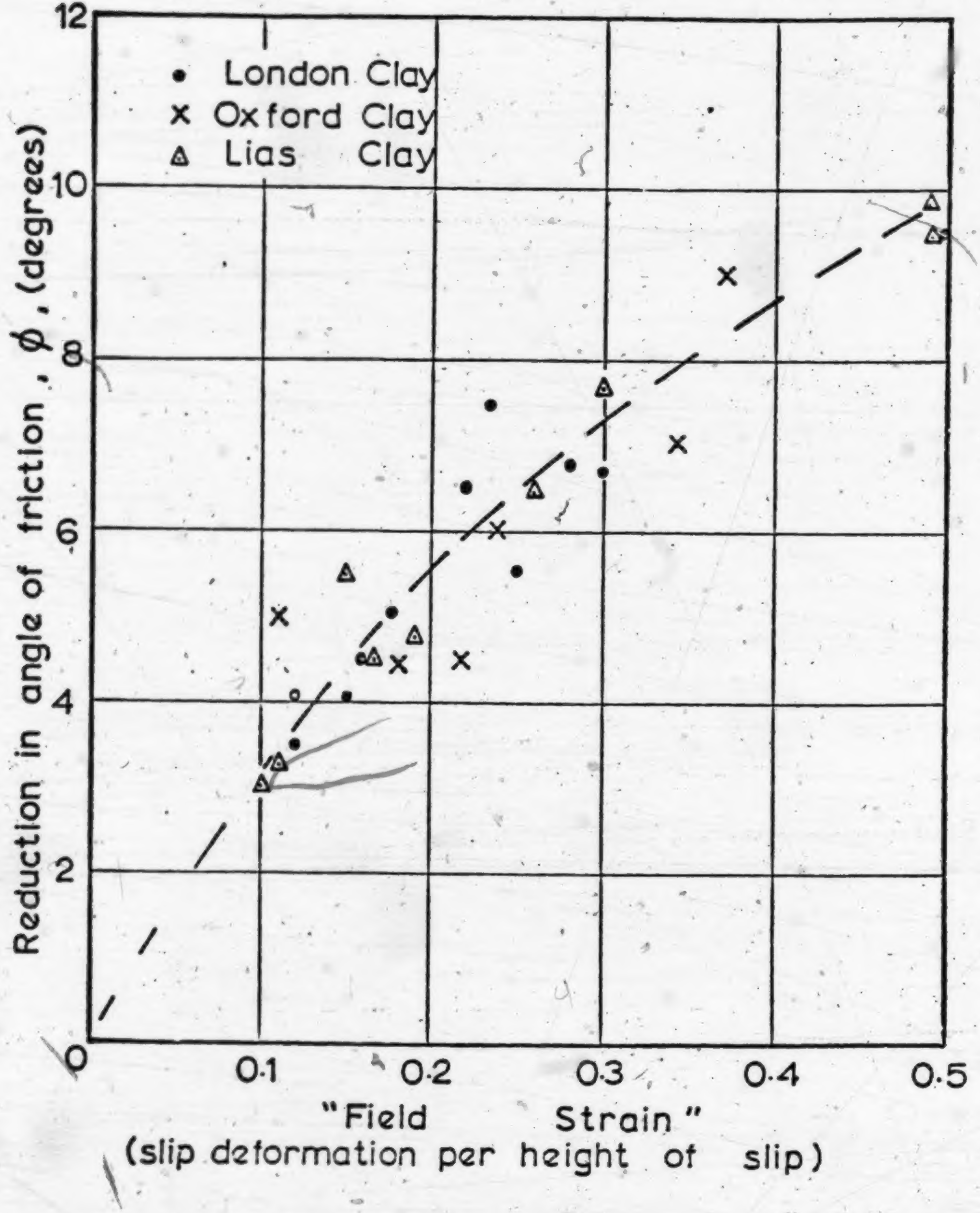
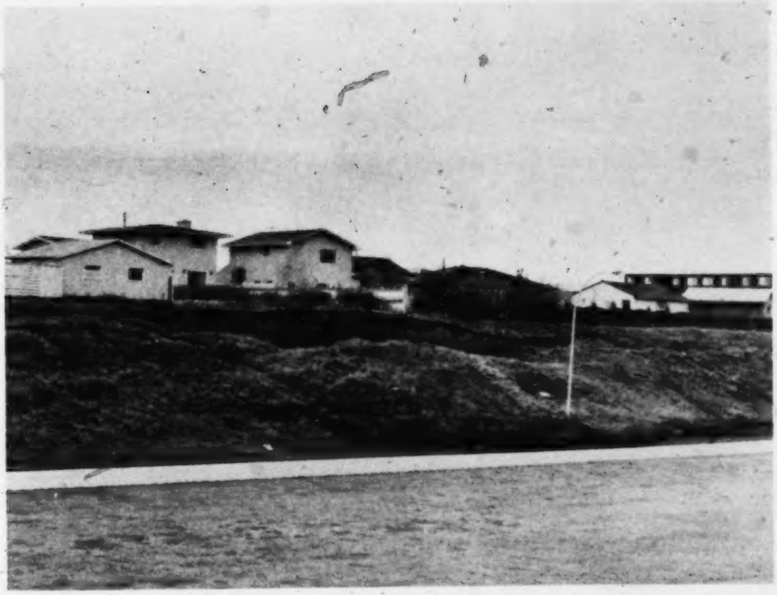
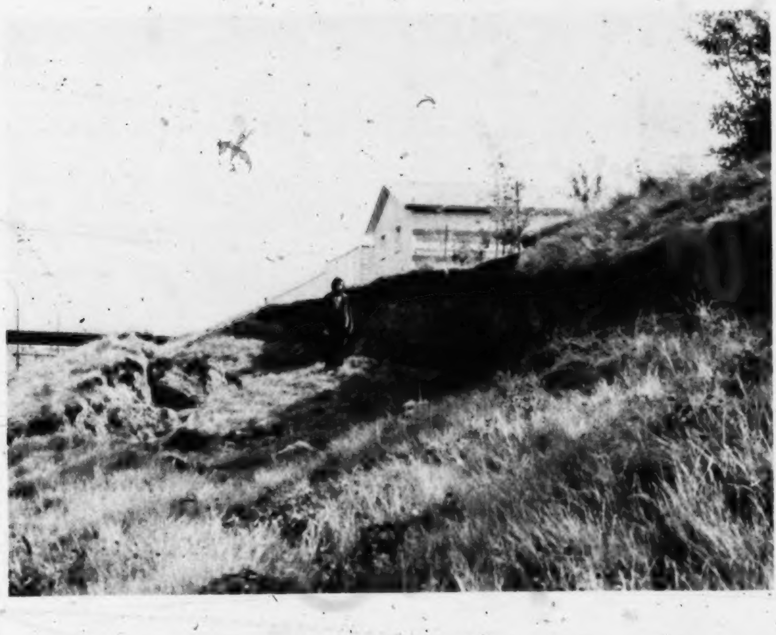


Figure 1.2 Relationship Between Movement of Slip and Loss of Strength (After James, 1970)

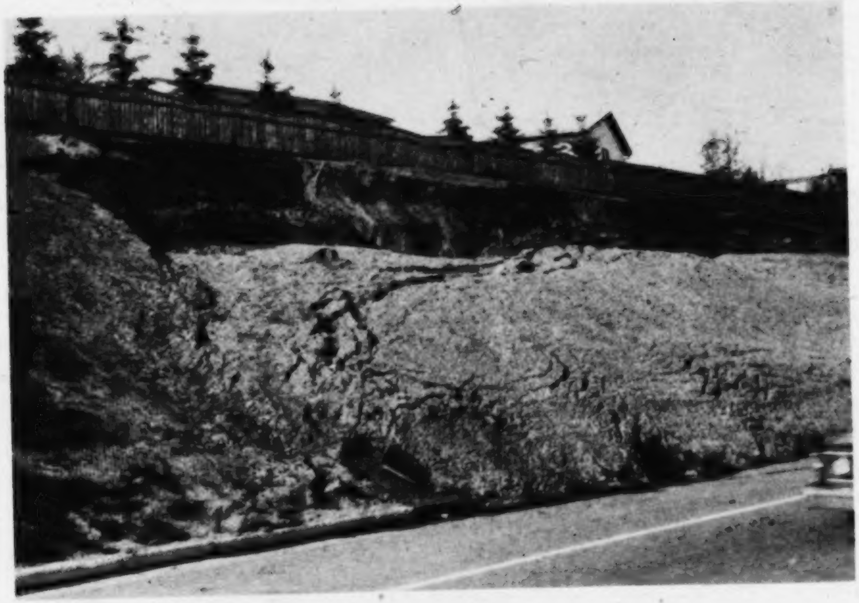


1a General view looking southerly.



1b View of scarp area showing the graben.

Plate 1 The 1973 Slide on the Whitemud Freeway.



2a Toe area and backscarp;note the seepage at the toe.

2b General view looking southerly;note stabilization operations at the west flank.

Plate 2. The 1976 slide on the Whitemud Freeway.

CHAPTER II

GEOLOGIC HISTORY, SITE DESCRIPTION AND SITE INVESTIGATIONS

2.1 Geologic History

The major stratigraphic units of the Edmonton area comprised the Upper Cretaceous bedrock, consisting of the Belly River, Bearpaw and Horseshoe Canyon Formations overlain by the Tertiary Saskatchewan Sands and Gravels. The Horseshoe Canyon Formation, which is the youngest of the three bedrock units, is composed primarily of interbedded and intertonguing sandstones, siltstones, shale and coal. Bentonite is present both as a constituent mineral in the clastic rocks and as individual beds (Irish, 1970). The sands and gravels which unconformably overlie the bedrock, are found within preglacial valleys as terrace deposits and valley-fill deposits (Westgate, 1969).

The preglacial topography was generally similar to that of today, with the exception of the glacial and post-glacial modifications to the preglacial drainage system, viz., infilling of preglacial channels with glacial materials and the subsequent incision of new channels (Kathol and McPherson, 1975).

The advance of the continental ice sheets resulted in deposition of a clay till, which is referred to locally as the "Lower Till" (Figure 2.1). Overlying this are discontinuous beds of stratified sands and gravels, in places up to forty feet (12.1 metres) thick, termed the Tofield Sands (Westgate, 1969). These are overlain by a sand till referred to informally as the "Upper Till". Both till units have extensive joint systems; the upper displaying vertically oriented, columnar joints as opposed to the rectangular joint system of the lower till unit (May and Thomson, 1977).

The stagnation of the last ice sheet in the area resulted in the formation of proglacial Lake Edmonton. Sediments deposited during the existence of this lake include a sequence of interbedded and intertonguing silts, sands and clays overlain by varved clay and silt (Hughes 1958, May 1977). Development of the lake is believed to be due to the ice blockage of drainage on the eastern, and probably the northern and souther shorelines. Hughes (1958) has divided the history of glacial Lake Edmonton into two stages.

Stage I involved the steady expansion of the lake as ice on the shores and ice-islands in the lake melted. Hughes (1958) has postulated the lake to be of superglacial origin, which after continued melting of the ice came to lie directly on the underlying till. The sediments were transported into the lake from a number of sources, viz.:

1. Meltwater streams carried debris from the

P

downwasting ice masses into the lake. The coarse fractions were deposited on the margins while the fines were carried to the center of the basin.

2. Icebergs and shore-ice rafted material into the lake.

3. A reworked pitted, deltaic deposit which was built into the lake from the west provided silts and sands to the basin (Figure 2.2; unit 3a represents the remnants of this delta). This is believed to have been the main inlet into Lake Edmonton during Stage I.

The lowermost units of the "Lake Edmonton Sediments" are predominantly well-bedded, poorly sorted sands (Figure 2.1) which have been penecontemporaneously deformed into folds (Westgate, 1969). The predominantly sandy composition reflects the relatively high energy of the meltwater in the initial stages of the formation of the lake. As the lake filled to its maximum depth, which is believed to be about 365 feet (Hughes, 1958), a sequence of interlaminated buff silts and dark silty clays were deposited. The interlaminated sediments have been highly disturbed and contorted in some localities. The disturbance may be attributed to a number of phenomena, including:

1. Large, floating icebergs, which when blown across the lake scoured and churned up the sediments.

2. The deposition of ice-rafted pebbles, stones or chunks of frozen till, sands or bedrock disturbed the bedding when settling into the soft, unconsolidated sediments.

3. Submarine or subaerial slumps may have occurred upon seasonal drawdown of the lake level.

4. The slumping of superglacial material from glacial ice into the lake would have disturbed the lake floor. These deposits are similar to turbidites.

The damned-up meltwaters continued to fill the Lake Edmonton basin until an outlet was uncovered. Such an outlet was found in the southeastern section of the lake (Figure 2.2) and has been termed the Gwynne outlet. The subsequent partial drainage of the lake through this spillway marked the end of Stage I. The maximum aerial extent of the lake roughly corresponded to the unit marked "Lake Edmonton Plain" on Figure 2.2. The basin of the lake was trough shaped, with the long axis approximately parallel to the North Saskatchewan River system (Bayrock and Hughes, 1962).

The drainage of the lake through the Gwynne outlet was at first slow, as lacustrine silts and clays continued to be deposited in the region near the spillway. With time, the spillway incised into the underlying glacial sediments and bedrock and flow greatly increased (Bayrock and Hughes, 1962). This drainage stage was probably of relatively short duration, as the lack of beaches in the shoreline regression areas seems to indicate. The presence of ice-rafted materials in the youngest lacustrine deposits indicates the existence of shore-ice and icebergs until the complete drainage of the lake. The drainage through the Gwynne outlet was thought to have lowered the lake level about 100 to 150 feet, thus effectively draining most of the Edmonton area (Bayrock and Hughes, 1962). With further degradation of the ice in the northeast, the present North Saskatchewan River system developed and in time completed the drainage of glacial Lake Edmonton.

During the Altithermal period, 7,000 to 4,000 years ago, the climate of Alberta was much drier and warmer than at present; sand dunes formed to the southwest of Edmonton (Figure 2.2) and desiccation is thought to have occurred at least into the upper Till Zone (May and Thomson, 1977). In post-Altithermal times little modification of the general topographic features has occurred, with the exceptions of slope instability along the drainage courses and the physical and chemical weathering of the surficial soil zones.

2.2 Field Description of Soils

The site location and study area, which are depicted in Figures 1.1 and 2.3 respectively, are located in the central portion of the Lake Edmonton plain (Figure 2.2). An examination of the 1950 air photos (RCAF A12865, #53 and

#54) reveals the site to be situated on a topographically high, level area between two tributary drainage networks of the North Saskatchewan River. Morphological features of the underlying till are faintly discernible through the lacustrine mantle deposits.

A field investigation of the study area (in particular, the backscarp of the 1976 slide as noted on Plate 2A, Chapter I) was undertaken in the spring and summer of 1977. The compositional heterogeneity and structural complexity of the deposit obviated the possibility of defining a "type-section" for the site, so compositional and structural peculiarities of the section are highlighted in the discussion that follows. Detailed stratigraphic mapping to the nearest centimetre would be needed to completely define the section.

The section generally displayed a mottled texture of buff silt and dark gray clay, with sporadic root channels and small, localized zones of selenite. Lenses of the dark gray clay up to about five centimetres thick were noted near the base of the section, at depths from about five to seven m. below the top of the scarp. A zone of discontinuous, irregularly shaped blocks and lenses of washed, medium-grained sand, each approximately 15 to 20 cm. thick was detected at a depth of 2.5 to 3.0 m. They persist for about three to four m. in lateral extent. The boundaries with the mottled silts and clays are sharp. This would indicate that the sands were frozen blocks when deposited.

Ice-rafted pebbles, cobbles, chunks of till, coal and sand (plate 3A) were found through the section at all elevations. This finding agrees with Hughes' (1958) observations.

The backscarp section could have been divided into two major zones on the basis of the macro-structure of the soil. The top section, which extended to a depth of approximately two to three metres below the top of the scarp is highly fractured and fissured, displaying what may be termed a blocky or nugget structure (Plate 3B). The size of the nuggets, which generally increases with depth, ranges from a few millimetres to about two to three centimetres. The boundary with the lower zone is transitional, extending over about one metre.

The lower zone does not possess the blocky or nugget structure of the upper zone. It is not, however, totally inact. A system of near-vertical, orthogonal joints has been mapped. The joints are usually no greater than about 1.5 to two metres high and could be traced for about one to two metres in lateral extent. One set (consisting of seven measurements) has an average strike of 260° , with variation from 237° to 303° . The intersecting set trends at about 153° . The joints were typically moist and gleyed with undulatory and hummocky surface expression. The surfaces, however, were not "softened" to any great degree.

The origin of the joint sets may be attributed to two possible mechanisms:

1. Isostatic rebound of the bedrock in the post-glacial period may have imparted tha columnar and orthogonal joint sets to the upper and lower tills. It is not unreasonable to propose further propagation of these structural elements into the lake sediments as well. The structural trends measured in this investigation roughly correspond to structural elements of this region as measured by Ozoray (1972).
2. The intense desiccation during the Altithermal period could have resulted in the formation of the joint sets. Quigley (1975) and Mitchell (1976) have discussed this process of joint formation in some detail.

A fissure network in the bands and lenses of the dark gray clay was observed both in the field and particularly in the laboratory specimens (section 3.4.1, Chapter III). The fissures were random in orientation, dip (range from horizontal to about 65°), and spacing (spaced as close as two to four millimetres in some zones). The surfaces were usually curvilinear convex or concave and undulatory with a hummocky texture; the lack of well-defined striations suggests that little or no relative movement has occurred.

The fissures were rarely greater than about 10 cm. in extent. Examination of the laboratory specimens (section 3.4.1, Chapter III) demonstrated that these fissures are confined to the dark gray clay lenses and are not found in the buff silt; in fact, fissures in the dark gray clay ended abruptly at the clay/silt boundaries. This finding is in agreement with Marsland and Butler (1967), who stated:

> "...Clay which contained more than 60% of fine particles was usually found to be highly fissured while that containing a lower percentage was usually unfissured."

The dark gray clay was found to contain about 77% of fine particles, while the buff silt contained only about 25% (Table III.1).

As Morgenstern (1967) has indicated, the mode of origin of fissures remains obscure. Two possible causes of the fissures found in the dark gray clay lenses and bands are postulated below:

1. Desiccation, such as that associated with the Altithermal period, may have resulted in the generation of the fissures. Differential desiccation, which is here defined as the simultaneous desiccation of sub-layered materials with markedly differing shrinkage characteristics could account for the random orientation versus the more vertical inclination of typical shrinkage cracks.
2. The Lake Edmonton stratigraphic column could have

been completely frozen in periglacial times, when the retreating ice was still in close proximity. The fissures, therefore, may be remnants of selective periglacial ice lensing in the dark gray clay-rich material.

Fookes (1965), Esu (1966) and Skempton et al (1969) have indicated that fissures or joints of tectonic origin or associated with shearing processes generally show a preferred orientation, while those due to physical and chemcial weathering or desiccation processes are usually randomly oriented. These observations tend to support the theories postulated above, rather than a tectonic or induced shear origin.

In summary, the main points which the preceding discussions have emphasized are the compositional heterogeneity and structural complexity of the Lake Edmonton Sediments in which the slope failures have taken place. The heterogeneity has been attributed primarily to processes associated with the deposition of the materials, such as disturbances due to icebergs, ice-rafted materials or slumps and mudflows. The structures, on the other hand, can be more readily related to diagenetic processes such as physical weathering and desiccation or isostatic rebound.

2.3 Site Investigations

2.3.1 General

The site investigations on the Whitemud Freeway cut pertinent to this thesis research were undertaken on two separate occasions:

1. During the fall of 1973, a detailed survey and subsurface exploration program was conducted on the 1973 slide by graduate students of the Department of Civil Engineering.
2. During the spring and summer of 1977, an extensive soil sampling program was undertaken by the author in the vicinity of the 1976 slide.

The results of both investigations, including landslide plans and profiles, borehole information and sample locations are presented in the following subsections.

2.3.2 Investigation of the 1973 Slide

The location and nature of the 1973 slide are depicted on Figure 2.3 and Plate 1 respectively. As mentioned in Chapter I, the slope failures in the Whitemud Freeway cut have been along non-circular slip surfaces. One of the main objectives of the investigation conducted in 1973 was to define the location of the slip surface precisely; this was accomplished by a series of six boreholes and five toe trenches, subsequently tied together with stadia surveying.

A detailed plan of the 1973 slide with the location of the profiles, boreholes and toe trenches is presented in Figure 2.4. The profiles and the location of the the slip surface, as interpreted from visual offsets and cavities in the boreholes and direct observations in the toe trenches are depicted on Figures 2.5, 2.6 and 2.7.

2.3.3 Investigation of the 1976 Slide

The 1976 slide (Figure 2.3), which is in part a reactivation of the 1974 failure, was repaired in the spring and summer of 1977 by the City of Edmonton. The primary method of stabilization involved the installation of counterfort drains. The soil sampling at the site was undertaken in conjunction with this stabilization program.

It was at first thought that block sampling would provide the best quality, undisturbed specimens for the laboratory testing program, but this was not the case. The samples in the upper zone were carved with little difficulty, but upon transportation from the site they literally "fell apart" along the structural discontinuities. A tube sampling procedure utilizing 10.2 cm. diameter pitcher sampler tubes was considered the best alternative.

The method of sample acquisition was as follows:

1. The first step was to excavate a level portion or a bench into the material to be sampled, whether it be in the base of one of the trenches or the backscarp.
2. After the bench was suitably prepared and leveled, "starter or guide holes" were drilled for the tubes. A portable, gasoline powered auger was used for this operation (Plate 4A). The holes were usually drilled about 10 cm. deep on approximately 20 cm. centers.
3. The tubes, being properly positioned in the starter holes, were thrust vertically into the bench with the shovel of the backhoe as shown in Plate 4B.
4. Recovery of the tubes was accomplished by either wrapping a chain around the end and extracting them with the backhoe or by simply excavating the bench. Upon recovery, the tubes were immediately transported to the Soils Preparation Room at the University. The samples were at that time extruded, waxed and placed in a moisture controlled room until testing.

The locations of the samples used for the testing program are depicted on Figure 2.3 and Figure 2.8.

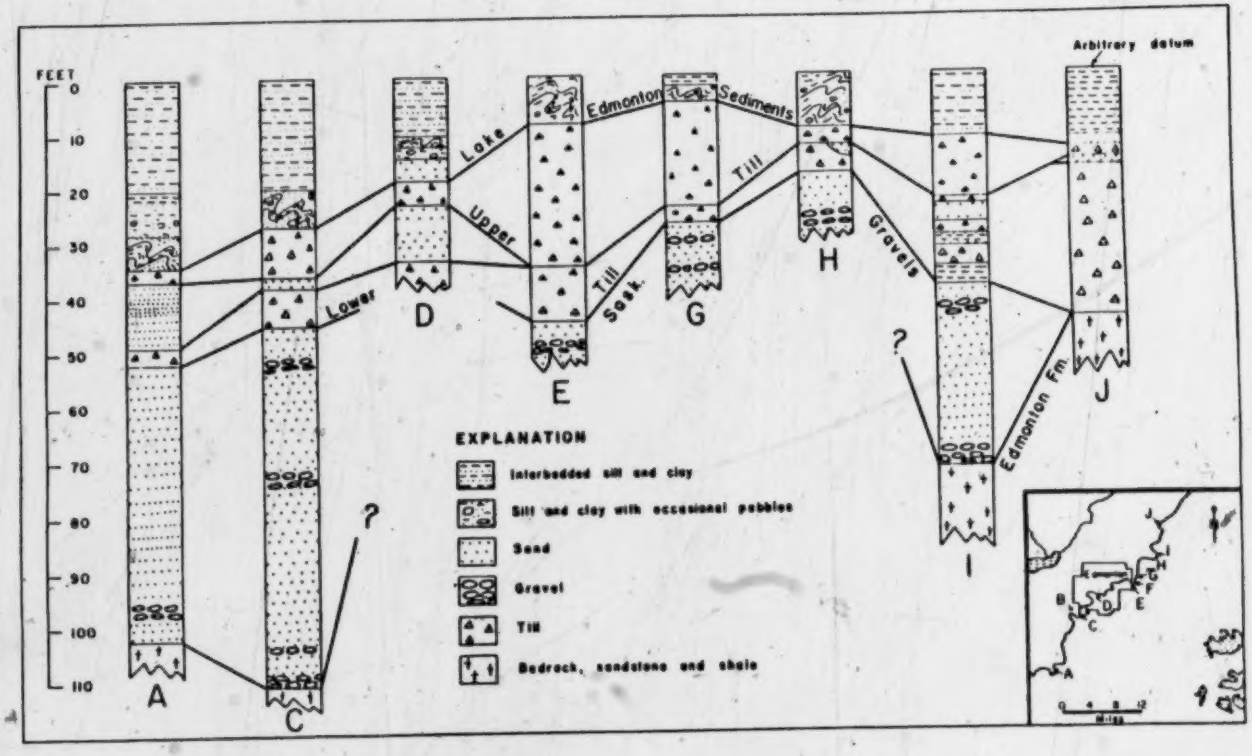

FEET
0
10
20
30
40
50
60
70
80
90
100
110
A
C
D
E
G
H
I
J
Arbitrary datum
Lake
Edmonton
Sediments
Upper
Till
Lower
Sask.
Till
Gravels
Edmonton Fm
?
?
EXPLANATION
Interbedded silt and clay
Silt and clay with occasional pebbles
Sand
Gravel
Till
Bedrock, sandstone and shale
A
B
C
D
E
F
G
H
I
J
0 4 8 12
Miles

Figure 2.1 Stratigraphy of Pleistocene Deposits Exposed Along North Saskatchewan Valley (After Westgate, 1969)

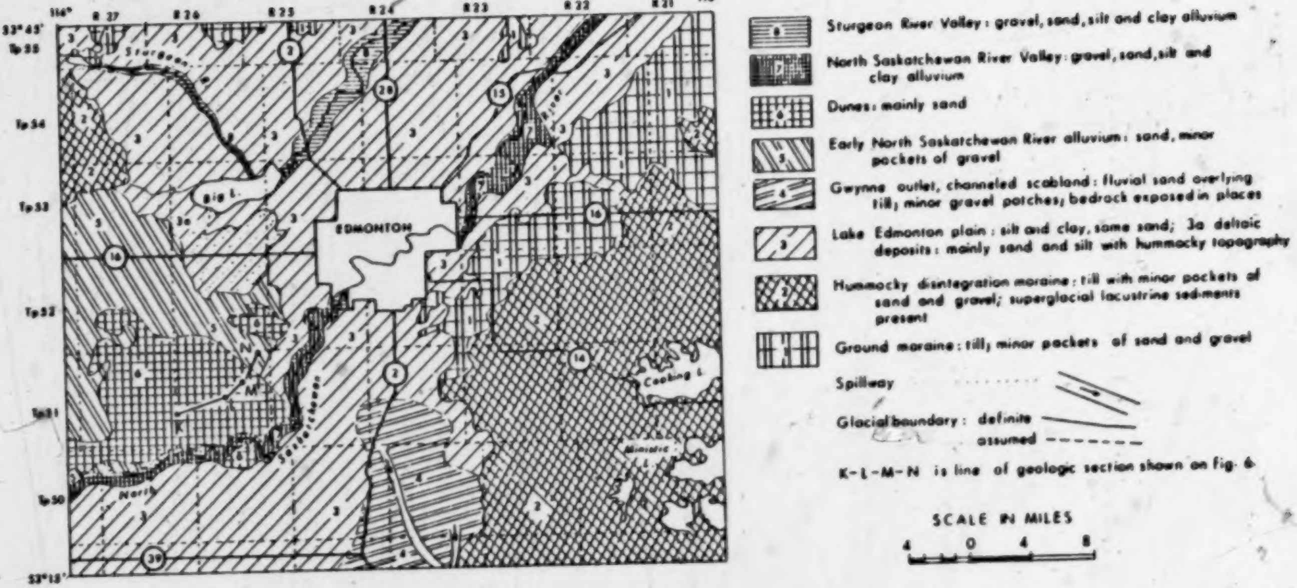

116°
R 27
R 26
R 25
R 24
R 23
R 22
R 21
113°
53°45'
Tp 55
Tp 54
Tp 53
Tp 52
Tp 51
Tp 50
53°15'
EDMONTON
Big L.
Sturgeon
North
Saskatchewan
River
Cooking L.
Miquelon L.
K
L
M
N
8 Sturgeon River Valley: gravel, sand, silt and clay alluvium
7 North Saskatchewan River Valley: gravel, sand, silt and clay alluvium
6 Dunes: mainly sand
5 Early North Saskatchewan River alluvium: sand, minor pockets of gravel
4 Gwynne outlet, channeled scabland: fluvial sand overlying till; minor gravel patches; bedrock exposed in places
3 Lake Edmonton plain: silt and clay, some sand; 3a deltaic deposits: mainly sand and silt with hummocky topography
2 Hummocky disintegration moraine: till with minor pockets of sand and gravel; superglacial lacustrine sediments present
1 Ground moraine: till; minor pockets of sand and gravel
Spillway
Glacial boundary: definite
assumed
K-L-M-N is line of geologic section shown on fig. 6
SCALE IN MILES
4
0
4
8

Figure 2.2 Quaternary Landforms and Deposits of the Edmonton District, Alberta (After Westgate, 1969).

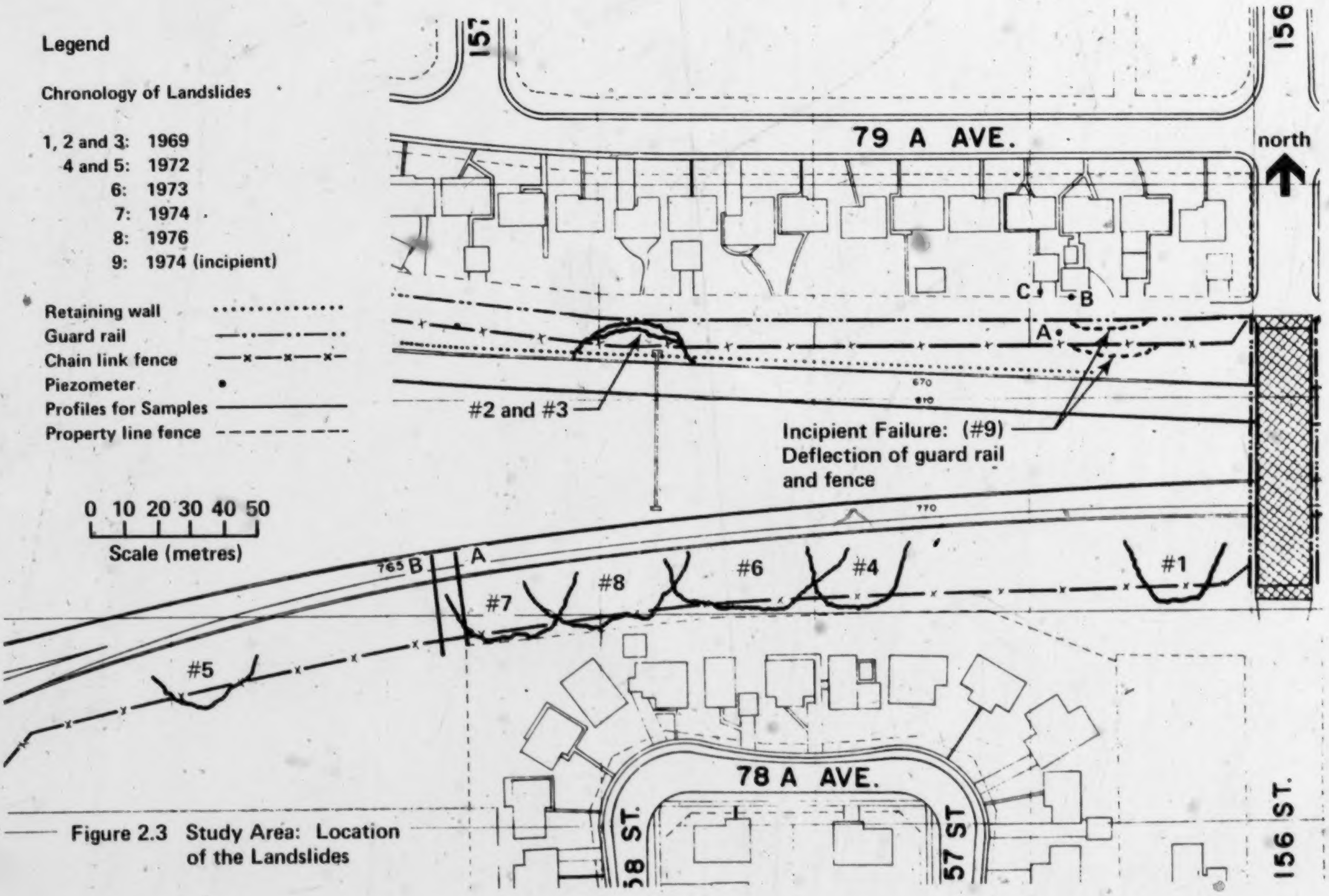

Legend
Chronology of Landslides
1, 2 and 3: 1969
4 and 5: 1972
6: 1973
7: 1974
8: 1976
9: 1974 (incipient)
Retaining wall
Guard rail
Chain link fence
Piezometer
Profiles for Samples
Property line fence
0 10 20 30 40 50
Scale (metres)
157
156
79 A AVE.
north
C
B
A
#2 and #3
Incipient Failure: (#9)
Deflection of guard rail
and fence
670
770
765
B
A
#1
#4
#6
#8
#7
#5
78 A AVE.
58 ST.
57 ST
156 ST.

Figure 2.3 Study Area: Location of the Landslides

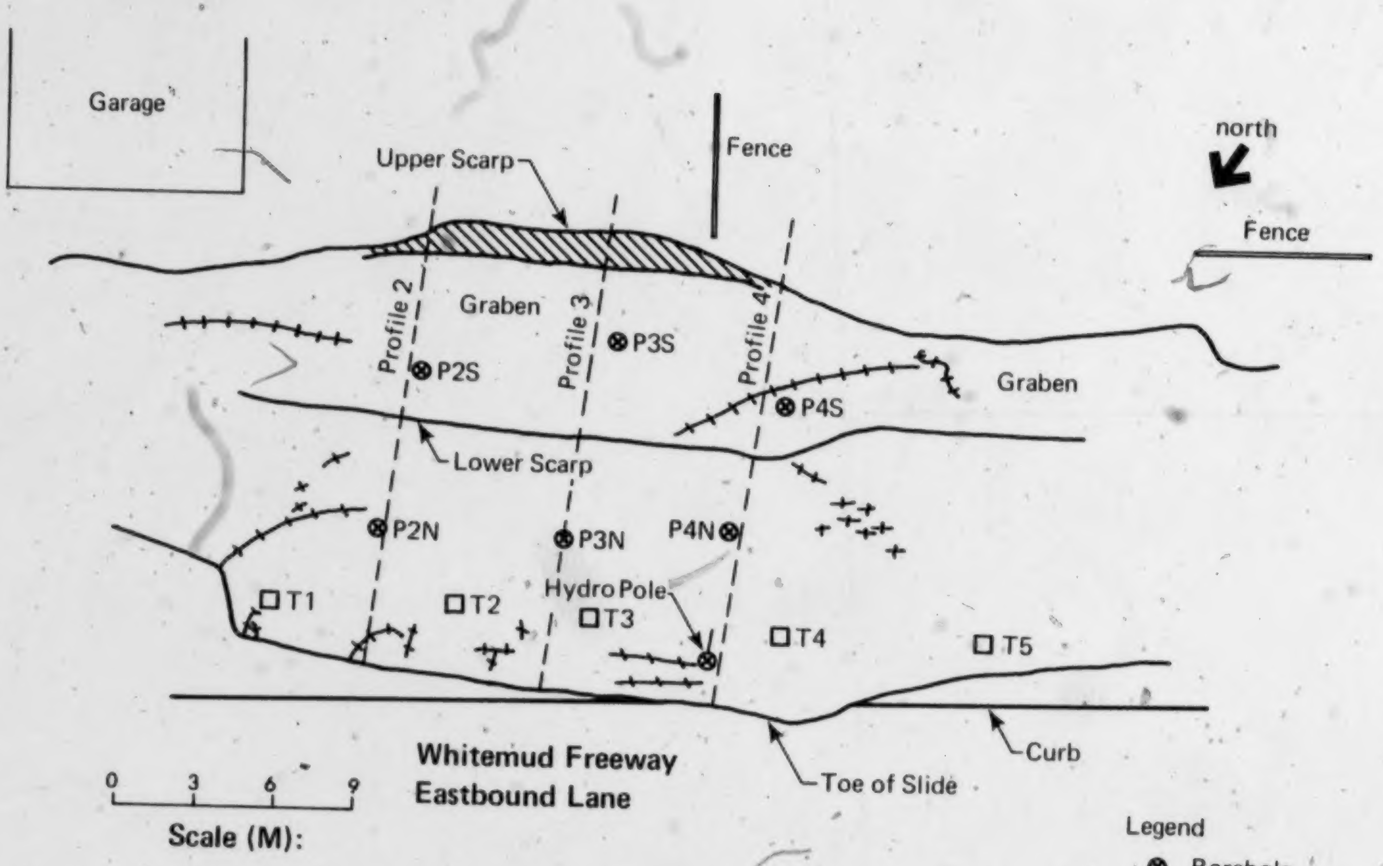

Garage
Upper Scarp
Fence
north
Fence
Profile 2
Graben
Profile 3
Profile 4
P2S
P3S
P4S
Graben
Lower Scarp
P2N
P3N
P4N
Hydro Pole
T1
T2
T3
T4
T5
Whitemud Freeway
Eastbound Lane
Toe of Slide
Curb
0
3
6
9
Scale (M):
Legend
⊗ Borehole
□ Test Pit
Crack

Figure 2.4 Plan of the 1973 Slide

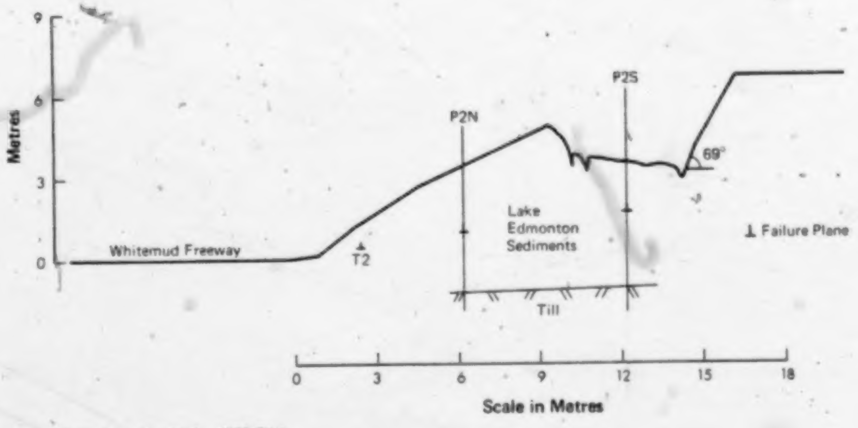

9
6
3
0
Metres
P2N
P2S
69°
Lake
Edmonton
Sediments
⊥ Failure Plane
Whitemud Freeway
T2
Till
0
3
6
9
12
15
18
Scale in Metres

Figure 2.5 Profile 2 1973 Slide

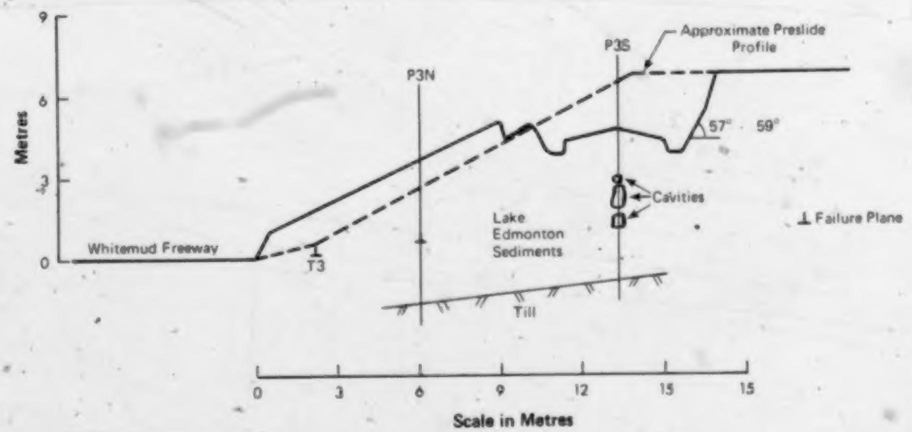

9
6
3
0
Metres
Approximate Preslide Profile
P3S
P3N
57°
59°
Cavities
⊥ Failure Plane
Lake
Edmonton
Sediments
Whitemud Freeway
T3
Till
0
3
6
9
12
15
15
Scale in Metres

Figure 2.6 Profile 3 1973 Slide

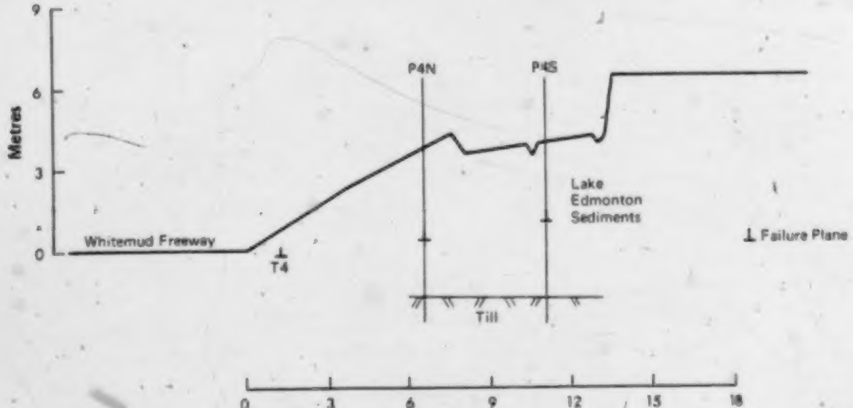
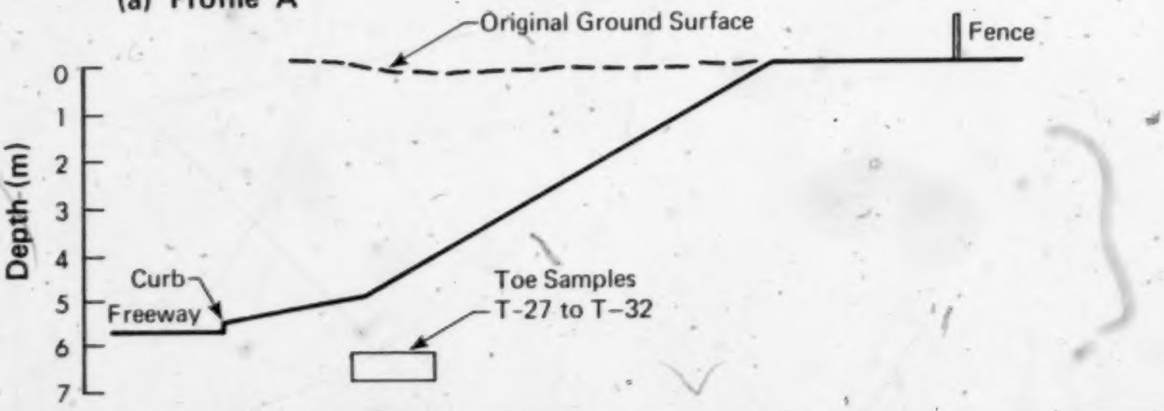

9
6
3
0
Metres
P4N
P4S
Lake
Edmonton
Sediments
⊥ Failure Plane
Whitemud Freeway
T4
Till
0
3
6
9
12
15
18
Scale in Metres

Figure 2.7 Profile 4 1973 Slide

(a) Profile A


Original Ground Surface
Fence
Depth (m)
0
1
2
3
4
5
6
7
Curb
Freeway
Toe Samples
T-27 to T-32

(b) Profile B

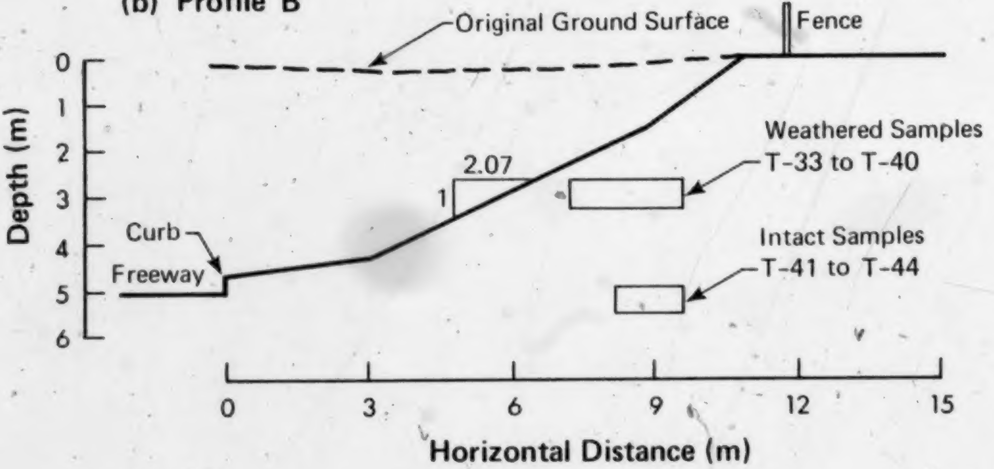
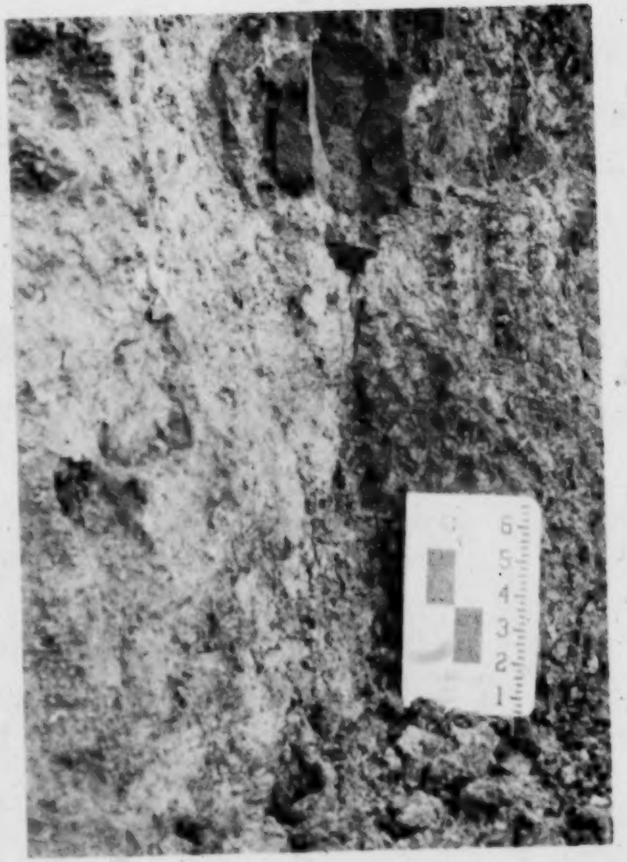
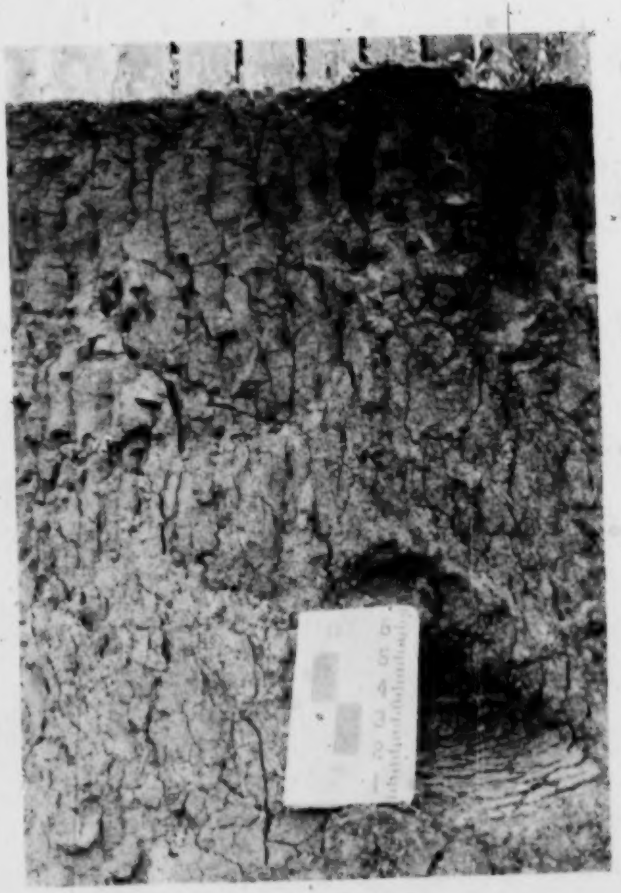

Original Ground Surface
Fence
Depth (m)
0
1
2
3
4
5
6
Curb
Freeway
2.07
1
Weathered Samples
T-33 to T-40
Intact Samples
T-41 to T-44
0
3
6
9
12
15
Horizontal Distance (m)

Figure 2.8 Sample Locations



3a Joint surface(from left to center of photo);note the dark block of sand.



3b Blocky structure due to desiccation near the surface.

Plate 3 Structures in the Lake Edmonton Deposit : Backscarp of the 1976 Slide.

4a Preparing guide holes for the sampling tubes.

4b Pushing a tube with the backhoe shovel.

Plate 4 Sampling Operations- The 1976 Slide.

CHAPTER III

LABORATORY TESTING PROGRAM

3.1 Introduction

The results and interpretations of the various laboratory tests conducted for this research project are presented in this chapter. The materials tested were obtained from three specific sample locations, as was outlined in Chapter II (Figure 2.8). For the purpose of discussion, tube samples T-33 to T-40 will be designated as "Weathered", T-41 to T-44 as "Intact" and T-27 to T-32 as "Toe" samples. The shear strength test results are grouped according to this classification.

3.2 Index Tests and X-Ray Diffraction Analyses

Due to the extremely complex nature of the soil, two distinct components, namely the dark gray silty clay and the buff clayey silt were separated and tested individually. For ease of discussion, the dark gray silty clay will be referred to as "clay", "dark gray clay" or "clay-rich material" and the buff clayey silt as "silt", "buff silt" or "silt-rich material". These connotations are based solely on the grain size analysis (Table III.1), as results of the Atterberg limits identify the dark gray material as a "CH" and the buff material as a "CI".

The majority of the dark gray clay sample was obtained from the failed sample of Triaxial Test #2 (sample T-43b); additional material was collected from Tube Sample T-44 in the vicinity of the samples of Direct Shear Tests #3 and #4 (Samples T-44a and T-44b). The clay was highly fissured, with fissures spaced as close as two millimetres in some instances, and displayed a laminated or leafy structure.

The buff silt sample was taken from Tube Sample T-43, directly below the material from which the sample of Direct Shear Test #2 (Sample T-43b) was carved. This silt-rich material displayed such structures as contorted varves and conchoidal fracture patterns.

As was expected, the clay-rich and silt-rich samples yielded markedly different index properties. The results of the Atterberg limit, hydrometer and specific gravity tests are summarized in Table III.1 (moisture content determinations for all the shear strength samples are presented in Tables III.6 to III.8). Due to the variable amounts of dark gray clay and buff silt in most of the samples, an accurate determination of the Liquidity Index is impossible. However, a range from about 0.1 to 0.5 using clay-rich and silt-rich samples respectively can be assumed.

Also included in Table III.1 are the results of X-Ray Diffraction tests performed on the clay-size fractions (i.e. finer than about two microns) of the two samples.

Mineralogical composition determinations, using this X-Ray Diffraction technique, are to be considered accurate to ± 10 percent.

3.3 Consolidation Test Results

A series of three consolidation tests were conducted on material from a nearby location at the site of the 1976 slide. The tube sample was from a depth of about three metres (the sample location, at the east flank of the slide, is depicted on Plate 4b). The oedometer samples were carved by carefully pushing a sharpened mold into the soil and trimming the excess as the mold advanced. The samples tested were 6.335 centimetres in diameter by 1.58 centimetres high. Typical results of one of these tests are presented in Table III.2.

The average coefficient of consolidation for the primary loading cycle to 96.6 kPa is 5.0 x 10^{-4} cm^2/sec. The average swelling coefficient determined from the unloading stages is 1.11 x 10^{-4} cm^2/sec. Using Casagrande's technique for determining the maximum preconsolidation pressure, a value of 72 kPa was obtained, which compares with an original overburden pressure of approximately 53 kPa. Therefore, an overconsolidation ratio of about 1.4 is indicated.

3.4 Shear Strength Study

An extensive shear strength testing program was undertaken to define the effects of physical weathering on the deposit, to try to put a time scale on this weathering and to check , through stability analyses, how these processes may affect the stability of the cut since its inception. For these reasons, the materials designated as "Weathered", "Intact" and "Toe" were chosen for testing. The following subsections will present brief sample descriptions and textural and structural differences between the three groups of samples, methods of sample preparation and procedures for performing the triaxial and direct shear tests, methods of interpretation of the raw test data and finally the test results. Interpretation of the results will be presented in a subsequent subsection.

3.4.1 Sample Descriptions

Although the deposit is very complex, with both facies and stratigraphic correlations being very difficult (as was outlined in Chapter II), there are certain distinctions both compositionally and structurally which may be made among the three groups of samples tested.

The material designated as "Intact" is, for the most part, composed of the dark gray clay and the buff silt described in the Index Tests subsection, with, in addition, localized sand pockets and isolated pebbles. The interrelationship of these components is so complex that no two samples would be alike, no matter how small the reference scale. Therefore, peculiarities of these samples will be discussed in a gross sense rather than trying to present a meticulous description of each sample.

Plates 5a and 5b depict two Intact 10.2 centimetre diameter triaxial samples. Upon first inspection, the heterogeneity of the material becomes readily apparent. The overall texture may be described as mottled or marbaloid, with lighter silt lenses or pockets surrounded in a darker clay matrix, or vice-versa. Upon closer inspection, features which reflect the highly disturbed and complex history become apparent. Contorted laminations with complex kink bands can be noted near the base of sample T-42a. The boundaries between the two soil types are erratic but very sharp. Flame and load cast type structures of the dark gray clay in the lighter buff silt may also be noted in the middle to top portion of sample T-42a. In sample T-44a, a general horizontal to sub-horizontal banding of silt and clay may be detected, but this too has obviously been highly disturbed. Erratic pebbles and sand pockets up to one centimetre in size have been detected in samples as well. Apart from textural and compositional irregularities, important structural features in the Intact samples may be noted.

The clay is highly fissured, with spacing in some locations as low as a few millimetres. The fissures are random in orientation, with dips ranging from horizontal to about 65°. Most of the fissures are less than five centimetres in extent, and very few are continuous across the entire 10.2 centimetres of sample. Micro-fissures have also been noted in the dark gray clay portions of clay/silt laminations only a few millimetres thick. The fissures always terminate at the dark gray clay/ buff silt interface. The fissure surfaces themselves are vitreous and undulatory, with hummocky micro-asperities. Convex fissure surfaces are also not uncommon. The fissure surfaces are generally not striated, but have a micro-hummocky type texture, which indicates that little or no movement has taken place along them.

A leaf-like or laminated type structure has also been noted in the dark gray clay, as discussed under the Index Tests. This imparts a structural anisotropy to the clay, which could result in a weaker shearing strength parallel as opposed to across the laminae.

The silt-rich material, except for a few random fractures and localized laminations, is massive and structureless. Fissure surfaces in the clay-rich material always end at the clay/silt interfaces. The fractures in the silt are rough and "conchoidal", characteristic of a brittle material. The laminations noted are quite contorted, displaying "S" folds and kink banding. The silty material, however, is the dominant component of the laminations.

Minor joint surfaces dipping at high angles (80°-90°) were detected in Triaxial Sample T-43C. The surfaces were gleyed and slightly hummocky, similar to those described in the Field Description of Soils section in Chapter II. These joint surfaces were not continuous across the the sample and were no greater than five to six centimetres in vertical extent.

It is apparent that the strength of the Intact samples will depend on the relative proportions of the silt-rich to the clay-rich material and especially on the location and orientation of the lenses and pockets of the fissured dark gray clay.

The Weathered samples, as the Intact, are all heterogeneous and display mottled or marbaloid textures (see Plates 5c and 5d). The silty dark clay occurs predominantly as seams and possibly remnant fissure infillings rather than in thick lenses and laminations. The gray clay/buff silt boundaries also are not quite as sharp, with diffusion of one material into the other. Very disturbed laminations also has been detected, and may be noted in the sample of Triaxial Test #9 as the sub-horizontal lighter and darker streaks. There were also numerous occurrences of isolated sand pockets and small pebbles in the samples. Orange oxidation zones were found to correspond with these coarser grained pockets.

The major difference between the Intact and Weathered samples, however, is in the structure. The Weathered material displays what may be termed a "nugget" structure; that is, the soil readily breaks down into little blocks or nuggets along randomly oriented discontinuities. The nuggets generally range in size from about five millimetres to two to three centimetres. The smaller nuggets occur in clay-rich areas of the specimens. The nugget surfaces are generally undulatory, convex or concave, very glossy and ridged. Each nugget may contain both the dark clay and the buff silt, so it appears that composition alone has not controlled the orientation and location of the nugget fractures. Plates 7c and 7d illustrate the nugget structure in the failed specimen of Triaxial Test #9.

The gleyed joint surfaces as noted for the Intact specimens also appear in the Weathered samples. The nugget structure though, is by far the dominant discontinuity in this material.

The Toe samples are characterized by sub-horizontal gray clay/buff silt banding, with bands ranging from about two millimetres to one centimetre in thickness. The bands are discontinuous in lateral extent, with the effect of having isolated lenses of buff silt in the dark gray clay and vice-versa. The contacts between the two materials are irregular, as with the Intact material, but are sharp in the sense that there is little diffusion of one material into the other. Thick lenses of the dark gray clay may also be found in this material, as was the case with the Intact, but not the Weathered. The clay bands and lenses for the Toe samples are characterized by a fine, blocky type structure on a scale of a few millimetres or less. The failed Triaxial Test #3c specimen, depicted on Plate 6a, shows this structure in the vicinity of the failure plane.

The Toe samples, as the other materials, contain sporadic pockets of fine to medium grained sand and isolated pebbles. Pieces of coal up to one centimetre in size have also been detected, but these probably have little effect on the strength.

In conclusion, therefore, the Toe and Intact samples are somewhat similar in their composition and texture, with both showing marbaloid or mottled textures and banding and laminations to some extent, but are quite different in the structure of the dark gray silty clay bands and lenses. The Toe material shows a much more pronounced blocky,aggregated structure than does the Intact material, which contains a series of random fissures. The Weathered material also displays the marbaloid or mottled texture which appears to be characteristic of the deposit as a whole, but does not have the banding or laminations to as great an extent as the other two materials. The Weathered material, similar to the clay-rich zones of the Toe material, has a very pronounced blocky or nugget structure.

3.4.2 Sample Preparation and Testing Procedures

Three types of shear strength tests were carried out on undisturbed samples in this investigation, viz.;

1. consolidated-drained triaxial tests
2. consolidated-undrained triaxial tests
3. consolidated-drained direct shear tests

The major advantages of the triaxial test are the possiblity of controlling drainage conditions and monitoring pore pressures in all phases of the test. There are, however, important limitations to this test, such as the influence of the intermediate principal stress being unaccounted for, being unable to model the effects on the pore pressures of rotation of the principal stress directions and the development of non-uniform stresses and strains across the sample. These and other factors are discussed in Bishop and Henkel (1962).

The reversing direct shear apparatus also has its advantages and limitations, which have been discussed by Chattopadhyay (1972). The major advantages are its ease of set-up and operation and the ability to subject samples to large displacements necessary for residual strength determinations. Notable limitations to the test are that the stress conditions in the sample are not precisely known and stress concentrations occur which result in progressive failure. The sample preparation and testing procedures for the three types of tests are described briefly in the following paragraphs.

The 10.2 centimetre diameter drained test samples were prepared by selecting an appropriate section of core and trimming it to the required dimensions. Due to slight lateral expansion in the 10.2 centimetre diameter sampling tubes, a few millimetres of material often had to be trimmed from the sides of the samples. This ensured, however, free radial drainage and helped to eliminate the disturbed radial areas. According to Bishop and Henkel (1962), the effects of friction between the ends of the specimen and the end caps, termed end restraint, may be partially overcome by testing samples with a ratio of height to diameter in the range of 1.5 to 2.5 diameters. For these tests, a ratio of two was chosen and deviation from this did not exceed about five percent in any test.

To reduce the time for consolidation from about 10 days to a more reasonable value of about one day, radial filter paper drains (Bishop and Henkel, 1962) were employed in all tests. Also, to eliminate leakage from the sample to the cell, two membranes were utilized in all 10.2 centimetre diameter tests. The use of these radial drains and membranes has an effect on the measured deviator load, and this effect will be discussed in the next subsection. The samples were set up in the triaxial cell using the standard procedure (Bishop and Henkel, 1962).

All samples were consolidated under a back pressure of 206.8 kPa to ensure complete saturation and to exclude the possibility of air locks forming in the drainage channels. An air lock would prevent the sample from taking on water during dilation and thus negating reliable volume change measurements.

After consolidation, appropriate strain rates were calculated (Bishop and Henkel, 1962). The rates used for these tests varied from 8.5×10^{-6} cm/sec to 1.3×10^{-6} cm/sec and are considered to be on the conservative side, so there would be no question as to the completeness of drainage and pore pressure dissipation during shear.

Following calculation of the strain rate, the triaxial cell was set up in the Farnell two ton capacity compression test machine and the sample was sheared at this rate well past the peak strength. Axial load cell and displacement transducer readings were recorded by a Hewlett-Packard data acquisition system, but volume change measurements had to be read manually from the burette. Since the tests usually ran for four to five days, the volume change readings could not always be taken at regular intervals. The missing readings were subsequently determined after completion of the test by interpolation from a curve of the recorded data points versus axial displacement. The volume change readings determined in this manner are considered to be within 0.5% of the "actual values".

The consolidated-undrained triaxial tests and three of the drained triaxial tests were performed on 3.8 centimetre diameter samples. The specimens for these tests were prepared by simultaneously pushing three 3.8 cm. diameter thin walled shelby tubes into a section of core and subsequently trimming them to the appropriate ratio of length to diameter of two diameters. These tests were set up in the same manner as the 10.2 centimetre diameter drained tests, but only one membrane was used, as the risk of leakage is proportionally less for the smaller samples.

The strain rates were again calculated using the method of Bishop and Henkel (1962) and the rates used for most tests were comparable to those for the larger samples. A backpressure of 200 kPa was used in all undrained tests, while a value of 206.8 kPa was used in the drained tests.

The axial loads and displacements were recorded by a Hewlett-Packard data acquisition system as previously. The drained test volume change measurements were again read manually from the burette. The pore pressures generated in the undrained tests were measured by means of a transducer and the readings were recorded with the axial loads and displacements on the data acquisition system.

The consolidated-drained direct shear specimens were carefully carved from the core samples by using a six centimetre square by 2.5 centimetre thick cutting mold. After the specimen was extruded from the mold into the Wykeham Farrance reversing shear box apparatus and consolidated under the applied normal load, the strain rate to peak was determined by substituting the coefficient of consolidatation calculated from the primary stage of the test into the formula developed by Gibson and Henkel (1954). The strain rates used in the tests varied according to the composition of the sample (e.g. the more silt-rich samples would have higher coefficients of consolidation and therefore, greater rates). Most samples, however, were sheared to peak stress at load rates between 2.0 X 10^{-6} and 8.0 X 10^{-6} cm/sec. These rates, similar to those used in the triaxial tests, are conservative and well within the recommendations of Cullen and Donald (1971) who suggest a rate of 4.2 X 10^{-5} centimetres per second for fissured, overconsolidated clays.

After the peak strength was passed the rate was increased by about four times to obtain the residual strength. This rate increase can be considered valid because the deformation rate formula is inversely proportional to the square of the drainage path, and after a shear plane has been formed (i.e. post-peak) the path is effectively cut in half. Kenney (1967) has shown that the residual strength is primarily dependent on mineral composition and not affected to any degree by strain rate.

3.4.3 Methods of Interpretation and Reduction of the Triaxial and Direct Shear Test Data

The triaxial tests have been interpreted and reduced following Bishop and Henkel (1962). A brief discussion of the important aspects of these procedures, for both the drained and undrained tests, will be presented in this subsection.

In the standard consolidated-drained triaxial test, the effective stresses are equal to the applied stresses. To calculate the deviator stress, the measured load must be divided by the effective or corrected cross-sectional area of the specimen at that stage. The expression used was as follows (Bishop and Henkel, 1962):

$$Ac=Ao(1+\triangle V/Vo)/(1-Sa)$$

where:
Ac = corrected cross-sectional area
Ao = initial average cross-sectional area after consolidation
$\triangle V$ = volume change at axial strain Sa
Vo = initial sample volume after consolidation

The consolidated-undrained test specimen was consolidated isotropically under a uniform cell pressure and then sheared under conditions of no drainage. Since excess pore pressures are measured in all stages of the test, the effective stresses can be calculated. The deviator stress was computed using the earlier expression for the corrected area, but with $\triangle V$ set set equal to zero.

In determining the peak strength parameters from undrained tests on dilatant materials, an ambiguity arises as to what is the failure condition. Bishop and Henkel (1962) have found that the stress ratio or obliquity peaks at a small fraction of the strain required to bring about the maximum deviator stress. The drop in pore pressure which comes immediately after the maximum obliquity results in the continued increase in the deviator stress. The strength parameters derived from the maximum obliquity have usually been found to be slightly greater than those from the maximum deviator stress, but the difference is usually of little practical significance. The maximum deviator stress criterion, therefore, is recommended because it gives a slightly more conservative estimate of c' and ϕ' and usually results in better separation of the Mohr circles and a more clearly defined failure envelope.

One aspect of the triaxial tests which has to be considered is the effect of the membranes and radial drains on the measured load at failure. Bishop and Henkel (1962) have quantified these effects and their findings are of interest.

In all tests on 3.8 cm. diameter specimens one membrane approximately 0.01 inches (0.025 cm.) thick was utilized. Extensive experiments have been carried out on the standard rubber membranes, including estimates of the compression modulus of the material, but all test results are with respect to plastic failure at 15 to 20% axial strain. For this failure mode, the correction was determined to be about 4.1 kPa at 15% axial strain. For a more brittle type failure on a single shear plane at much lower axial strains, (which was the mode of failure of the Lake Edmonton material), no satisfactory analysis is available. Through experience however, it is believed that the correction increases slightly with the cell pressure and at the lower failure strains the same correction of 4.1 kPa may be applied. (Symons (1967) has undertaken tests utilizing a perspex sample with a single shear plane to show that the correction does in fact decrease with the cell pressure. At a cell pressure of 105 kPa, the lowest tested, the combined membrane and drain correction was slightly less than 14 kPa at five percent axial strain).

A calculated relationship for 10.2 centimetre diameter samples which illustrates the variation in rubber membrane correction with axial strain is presented by Bishop and

Henkel (1962). For the four to six percent failure strain usually measured in tests on the Lake Edmonton samples, a correction of 0.1 psi (0.7 kPa) is obtained, which may be doubled for two membranes, giving a total correction of 1.4 kPa.

The filter drain correction is much more difficult to derive than that for the membrane alone, and the problem is compounded when brittle rupture on a single failure plane is considered. Available experimental data on 3.8 centimetre diameter specimens suggests that a combined membrane and drain correction of 13.8 kPa may be applicable to samples failing in the brittle mode. Bishop and Henkel (1962) believe that at cell pressures below about 35 or 40 kPa slip between the drain and sample occurs, thus reducing the correction. No experimental data on 10.2 centimetre diameter specimens has as yet been collected but the drain correction can be calculated based on the assumption that it is proportional to the 3.8 centimetre diameter sample correction. Therefore, taking the ratio of the sample diameters and multiplying by the 3.8 centimetre diameter correction yields a value of 3.4 kPa.

Therefore, from the preceding discussion using the limited amount of data available, a combined correction of 4.8 kPa and 13.8 kPa will be applied to the peak deviator stresses of the 10.2 centimetre and 3.8 centimetre diameter samples respectively. The validity of these corrections will be explored in a later section.

Morgenstern and Tchalenko (1967) reviewed current interpretations of the direct shear test. The so-called "common approach" is one which assumes that the shear strength is mobilized on the horizontal plane in the direction of the apparent shear stress, which is defined as the measured shear load divided by the sample area. If the material is cohesionless, the ratio of apparent shear stress to the apparent normal stress (the apparent normal stress being the normal load applied divided by the sample area) yields tanØ' directly. In the second approach, the conjugate slip lines are assumed to act at Ø'/2 and 90° - Ø'/2 to the horizontal. These slip surfaces indicate a condition similar to simple shear in the central portion of the specimen. If it is assumed that the principal stress axes coicide with those of pricipal strain rate, then the direction of the apparent shear stress is the direction of the maximum shear strain rate, and thus is the maximum shear stress. The ratio of apparent shear stress to normal stress now yields sinØ' directly. These interpretations, however, neglect end effects and assume constant volume deformation. Morgenstern and Tchalenko (1967) have discovered Riedel shear structures inclined at approximately Ø'/2 in specimens sheared to about the peak strength. These findings tend to support the second theory of simple shear conditions being set up in the central portion of the sample. Both interpretations will be considered in the tabulation of the results.

Aside from the interpretation of the stress state, one other factor must be considered. No area correction has been applied to either the apparent shear or normal stress calculations because according to Kenney (1967) both values are equally affected by the correction and thus the ratio of apparent shear stress to normal stress is unaffected.

3.4.4 Shear Strength Results

The consolidated-drained and undrained triaxial and direct shear test results are presented in tabular form in Tables III.3, III.4 and III.5 repectively. The effective stress path points at failure, failure strains and volume change and pore pressure conditions at failure have been included in the triaxial results while the horizontal displacements and vertical percent strain to peak have been incorporated with the direct shear strength results. Deviator stress versus axial strain and percent volume change versus axial strain curves for all the 10.2 cm. diameter drained triaxial tests may be found in Appendix A. (The plot titles are set up so that the number of the test comes first followed by the sample number, which actually indicates the tube sample from which the specimen was prepared. The values in paretheses are the cell and backpressure respectively). Typical undrained triaxial test curves of deviator stress, pore pressure and the A parameter versus axial strain and the total and effective stress paths have also been presented to demonstrate the undrained response of the material. These plots are included in Appendix A as well. For completeness, shear stress and vertical deformation versus horizontal displacement plots for key direct shear tests are also in Appendix A.

Tables III.6 to III.8 present pertinent "before and after test" properties of all the specimens tested. Included are the densities, moisture contents, void ratios and degrees of saturation, all of which assist in making valid interpretations and correlations of the strength characteristics of the material.

Mohr circle plots for all the triaxial tests are presented on Figures 3.1 to 3.4. They are grouped according to the type of sample (i.e. Intact, Weathered or Toe) and the type of test performed. Failure envelopes have been drawn giving certain tests more emphasis than others because of sample heterogeneity and correlations with direct shear data. This will be discussed in more detail in the "Interpretation of Test Results" section which follows.

The direct shear data has been plotted in Figure 3.5. The peak strength values at appropriate normal loads have been included. All the tests, however, did not reach what may be termed a reliable residual strength. Some tests were discontinued before similar results could be obtained on both the tension and compression cycles, as suggested by Cullen and Donald (1971). Other tests involve the post-peak extrusion of the "peak material" and shearing subsequently begins in a new and different layer. An example of this is shearing through the gray clay and into the buff silt portion of a varved specimen. As soon as the clay is extruded and shearing in the silt commences, the shear stress will increase noticeably (Direct Shear Test #2b, Appendix A).

Composite Mohr-Coulomb plots of the peak strength test results have been prepared (Figure 3.6, Figure 3.7). A discussion of the assumptions made and the reasons for defining the various envelopes for the Intact, Weathered and Toe materials may be found in the next subsection. Supporting evidence for this interpretation from the literature will also be included.

3.5 Interpretation of Test Results

3.5.1 Index Tests and X-Ray Diffraction Analyses

The results of the limited index tests and x-ray diffraction analyses undertaken for this research investigation are comparable to the data given by Thomson (1969). The grain size distributions presented by Thomson, with an average of about 35% silt and 60% clay sizes indicate a soil which is a mixture of the two components studied here. An average plasticity index of 41% for the mixture falls between the 61% and 21% for the dark gray silty clay and the buff clayey silt, respectively.

The dark gray silty clay (Table III.1) has a clay size fraction of about 77%, of which approximately 60% is montmorillonite. This high proportion of montmorillonite is reflected in the liquid limit of 98%. As Thomson (1969) pointed out the montmorillonite is believed to have originated from the glacial scouring of the montmorillonite (or bentonite) rich Upper Cretaceous bedrock during the advance of the continental ice sheets. The buff clayey silt, on the other hand, contains approximately 75% silt sizes, much of which is probably composed of the primary minerals such as quartz and feldspar (Dudas, personal communication). The surprisingly high plasticity of this material is due to the high proportion of montmorillonite in the clay fraction. The trace of interstratified mica-montmorillonite found in both samples demonstrates that the deposit has undergone significant weathering.

The main point which the index tests performed on the two completely different components has strengthened is the problem of sample heterogeneity and variability. A sample which fails predominantly through the buff silt-rich material would be expected to mobilize a much higher shear strength than one which fails through a clay-rich zone. In many cases failure has occurred through both constituents, and the relative proportions of silt-rich and clay-rich materials along the failure surface would be expected to strongly affect the strength of the specimen.

3.5.2 Consolidation Tests

The consolidation test sample was described by Mawhinney (1978) as a mottled, dark gray/buff silt-rich material displaying a slickensided or blocky structure. The specimen could , therefore, be considered to be "weathered". Irregular settlement was noted under the first load increment of 10.7 kPa, and could be partially attributed to readjustment along the slickensides. The coefficient of consolidation (Cv) for the first loading is much higher than the values under subsequent load increments; this may be partially due to the effects of fracture (mass or secondary) permeability. The overconsolidation ratio of 1.4 indicates slight overconsolidation , and this is in agreement with the theories of freezing and desiccation effects on soils which are discussed in the following subsection.

The results presented in Table III.2 are slightly higher than those of Thomson (1969), but the latter samples were remoulded and contained a higher proportion of clay. Coefficients of consolidation for the 10.2 centimetre diameter triaxial specimens ranged from 0.82 to 3.9 X 10^{-4}

cm²/sec with an average value of 2.4 X 10^{-4} cm²/sec, which agrees very well with the results shown in Table III.2. There was not a noticeable difference between the Intact and Weathered specimens.

There was a marked difference, however, in the swelling coefficients of the two materials. The 10.2 centimetre diameter Intact sample consolidated under 20.7 kPa effective consolidation pressure yielded a swelling coefficient Cs, of 0.85 X 10^{-4} cm²/sec, whereas the weathered sample under the same pressure swelled at 1.6 X 10^{-4} cm²/sec., nearly a factor of two higher.

It is believed, however, that the average swelling coefficient of 1.19 X 10^{-4} cm²/sec derived from the laboratory tests is not representative of the field conditions for the following reasons:

1. The large scale structural discontinuities which were noted as being open and thus having a high permeability in the field investigations (Chapter II) have not been adequately accounted for. Cedergren (1977) has pointed out that permeabilities may be increased by as much as one thousand times by structural dicontinuities.
2. Sand seams with a much higher permeability than the silts and clays have also not been accounted for.
3. The laboratory specimens undergo a much different

stress history than the material in the field. Unloading, then reconsolidation in the laboratory would tend to "close up" some of the discontinuities, while in the field the only mechanism is unloading, which tends to allow the discontinuities to open. Also, the action of sampling has created a smeared or disturbed zone around the periphery of the sample, which effectively closes or seals most of the fissures.

3.5.3 Shear Strength Tests

Upon initial investigation of the Composite Mohr Plot (Figure 3.6), the reader will have noticed that there is much scatter in the data. In order to define the various strength envelopes depicted on this plot, each test had to be considered individually and a decision made as to the mode of failure; i.e. whether failure occurred predominantly through the buff silty material or through the gray fissured clay, or through or around the nuggets. This subsection reviews the methods used and the assumptions made in constructing the Composite Mohr plot and then discusses stress-strain and volume change relations for the soils. Examples of similar behavior are cited from the literature as supporting evidence of the phenomena proposed.

As noted in the "Methods of Interpretation" subsection, both methods of interpretation of the direct shear test were considered. A much better correlation between the triaxial and direct shear data using the "common" rather than the "simple shear" approach was noted. Therefore, all direct shear data were plotted assuming the ratio of apprent shear to normal stress yielded tan Ø' directly (Figure 3.5).

Seven direct shear tests were undertaken on the Intact material. Of these, four define a linear envelope with a Ø' of 20.5° and an apparent c' of 15 kPa. Two are found to lie bove this envelope, with approxiamtely the same Ø' but having an apparent c' of about 25 kPa and one is found well below the envelope.

The four specimens on the failure envelope displayed a mottled or marbaloid texture of the buff silt-rich and gray clay-rich materials. The hummocky failure planes contained both the silt and clay in varied proportions. The silt-rich material had a torn or ragged, dull texture while the dark gray clay was smeared out and highly polished and slickensided.

Direct Shear Tests #1 and #2, which plot above the envelope contained more of the buff clayey silt and were denser than the other samples (Table III-8) These specimens were taken from tube sample T-43, directly above the material used for the index tests on the buff silt. The sample of Direct Shear Test #3 which plots below the envelope was composed predominantly of the fissured, dark gray clay. When carving the sample, a large, shallow dipping fissure was noted which would have been situated in approximately the middle of the specimen. This sample, therefore, reflects the lower strength of the dark gray, fissured clay-rich samples opposed to specimens containing more of the buff silt.

Using the "average" strength parameters of C'=15 kPa and Ø'=20.5° from the direct shear tests , the 10.2 centimetre diameter intact triaxial test Mohr circle plot was analyzed (Figure 3.1). Triaxial Tests #1, #4 and #5 yield circles which plot very close to the envelope. All three of these samples had the characteristic mottled or marbaloid texture of the buff silt and dark gray clay (Plates 5a and 5b). The failure planes were bi-angular, dual or uniform with dips ususally ranging from about 50° to 60° (Table III.6). Dual or parallel failure planes separated by about two centimetres occurred in Test #1. The failure surfaces, however, all passed through clay-rich and silt-rich zones, and as in the direct shear tests, the silt displayed a ragged, torn structure while the the clay was polished, slickensided and often smeared out over silt-rich zones.

Triaxial Test #2 falls well below the envelope, and the reason for this is that the failure was confined to a dark gray,fissured clay-rich lense near the top of the sample. Many randomly oriented fissure surfaces were noted just above and below the failure plane, and it is highly possible that fissures were involved in the failure. Triaxial Test #3, which plots above the evelope, failed predominantly through silt-rich material (Plate 6b). The top of the failure plane, where it passed through silt and clay, was polished and slickensided, with a smearing of the clay. The bottom portion of the failure surface, where it passed predominantly through silt, was rough and displayed a torn texture. The "tangent" points from these triaxial test Mohr circles were therefore transferred to the Composite Mohr Plot assuming a ϕ' of 20.5° in all cases.

The tangent points for the 3.8 centimetre diameter Intact triaxial tests were determined in the same manner, assuming a c' of 15 kPa and a ϕ' of 20.5° (Figure 3.2). All tests are in reasonable agreement with the envelope, except for Tests #1c and #1d. Test #1c was observed to have two failure planes at 60° and 45°, and, therefore, it is believed that structural discontinuities may have been involved in the failure. Test #1d could be somewhat low because of a substantial decrease in the membrane and drain correction below normal stresses of about 35 to 40 kPa, as discussed in Section 3.4.3. It can also be noted on the Mohr diagram that the undrained tests have slightly lower strengths than the drained tests. This effect has been attributed to the different energy relationships between the tests (Bishop and Henkel, 1962).

The Weathered direct shear test results are interpreted as giving a bi-linear envelope, with a ϕ' of 43° and an apparent c' of zero in the low normal stress range and a ϕ' of about 21° to 22° in the higher stress range (Figure 3.5). The break point in the envelope is at about 40 kPa, which closely corresponds to the overburden pressure of the samples (Figure 2.8, Chapter II).

Direct Shear Tests #5 and #6 were sheared under a normal stress above the break point, and gave typical failure surfaces as shown in Plate 6c. The surface was quite hummocky, with the ridges about four millimetres high. Such surfaces have been noted by Cullen and Donald (1971) and Tweedie (1976) and were attributed to hard inclusions or zones of higher strength in the sample. Direct Shear Tests #7 and #8 were performed with normal stress slightly above and below the break point respectively. Both tests, and particularly #8 (see Plate 6d) had failure surfaces with irregularly shaped cavities, as if nuggets had been plucked out and ground up under the shearing action. The typical nugget structure was seen in all samples, with nuggets rarely ranging down to about two to five millimetres in size. The nuggets were generally stiff, but soften slightly with decreasing size.

Direct Shear Test #10 (Figure 3.5, Table III.5), which appears anomalously high for the applied normal stress of

14.5 kPa, was prepared from a more intact section near the base of the Weathered tube sample T-39. This sample did not possess the nugget structure of sample T-38a of Test #8, but had, instead, a series of near vertical, gleyed fracture surfaces. It may be assumed that these dicontinuities were not active in the failure, and this specimen actually represents the strength of the intact material between the fissures.

Applying the bi-linear interpretation of the direct shear tests to the 10.2 centimetre diameter weathered triaxial test results and using a break point slightly above 40 kPa yields the strength envelopes as defined in Figure 3.3. The tangent points, using the ∅' angles indicated on Figure 3.3 have been plotted on the Composite Mohr diagram. Combining the direct shear and triaxial results yields the Weathered soil parameters of ∅' equal to 40° with a negligible c' below about 42 kPa effective normal stress and ∅' equal to 24.5° and c' equal to 17 kPa above the 42 kPa effective normal stress.

The modes of failure above and below the 'break point' (Figure 3.6) are different as is clearly shown by the photographs of the specimens of Triaxial Tests #6 and #9 (Plate 7). These samples were consolidated under cell pressures of 80.7 and 20.7 kPa, respectively. The failure surface of the specimen of Triaxial Test #6 was highly polished and slickensided except where the failure plane passed through silt-rich zones, where tearing and fracturing occurred (Plate 7a). The failure plane , which is "stepped" down in the direction of movement, appears to have followed the thin clay-rich seams in the sample. Smearing of the clay, as with the Intact specimens, also occurred. The other photographs of Plate 7 illustrate the sample of Triaxial Test #9. The failure surfaces were very irregular and neither polished nor slickensided to any great degree. It appears that failure occurred along the nugget discontinuities and not through the intact material. The sample literally "broke up" along the discontinuities.

The Toe material direct shear tests show much scatter (Figure 3.5). Tests "2a, #2b, and #2c closely parallel the Weathered direct shear test results, and this correlation seems to suggest a similar mechanism of failure. Tests #3b and #3c plot well below the other tests (figure 3.5), but are very close to the envelope defined by Triaxial Test #2 and Direct shear Test #3 (Figure 3.6). These latter tests failed predominantly through the dark gray silty clay, possibly involving fissures.

Upon re-inspection of the sample used for Test #3b, it was noted that the top half (above the failure plane) was almost all dark gray clay while the bottom half was composed almost entirely of buff clayey silt. It may be assumed, therefore, that failure occurred along the buff silt/gray clay boundary, which effectively acted as both a compositional and structural discontinuity. The horizontal diaplacement to peak is a factor of five times lower than any other test (Table III.5), indicating that "separation" along the discontinuity occurred before a peak resistance could be mobilized in either material. The sample from Test #3c was composed predominantly of the dark gray clay-rich material, and therefore has a strength similar to Triaxial Test #2 and Direct Shear Test #3. It is not known whether large fissure surfaces, such as those found in the Intact tests above or a nuggety structure too fine to give the bi-linear characteristic was operational in this specimen. The other specimens contained varying proportions of the nuggety dark gray clay and buff silt-rich material.

It can be assumed , therefore, from the limited direct shear data available that a bi-linear failure envelope best represents the failure modes of the Toe material.

The 3.8 centimetre diameter triaxial tests on the Toe samples are plotted on Figure 3.4. All six specimens tested had the characteristic banding or discontinuous lensing of the nuggety dark gray clay and buff silty materials as described in Section 3.4.1, but the width and extent of the bands and lenses varied from sample to sample. Failure took place through both the silt and clay, but the clay was usually predominant. Tests #3a and #3b, sheared under effective consolidation pressures of 70 and 50 kPa respectively, were observed to have smearing of clay and tearing through the silt on the failure planes, as with many of the Intact specimen failure planes. Test #2a, however, sheared under an effective consolidation pressure of 60 kPa, did not develop a well-defined shear plane through the clay-rich zone but rather seemed to fail in a compressional, more ductile mode. All tests sheared at less than 40 to 50 kPa effective consolidation pressures developed nuggety, rough failure surfaces and Plate 2, which depicts the Triaxial Test #3c specimen is a good example.

The use of a ϕ' equal to 40° in the low normal effective stress range, below about 42 kPa, and a ϕ' of 23° in the higher normal stress range, as developed for the direct shear test data, gives a very good fit to the triaxial data except for Test #2b, which is noticeably above the envelope (Figure 3.4). This specimen was discovered to have a leak in the top drainage connection, which would render it a partially drained test. As with the undrained Intact tests, the membrane and drain correction is believed to be much too high for the low normal stress Tests #2c and #3c. A lower correction would effectively shift the envelope up, maintaining the same ϕ' but giving a cohesion intercept closer to zero. The tangent points for these tests were determined using the friction angles as depicted on Figure 3.4, and plotted cn the Composite Mohr Diagram (Figure 3.6).

Using all the Toe tests in the low normal stress range and reducing the correction on Triaxial Tests #2c and #3c, a

ϕ' of 40° with a zero cohesion intercept could be defined, but the higher stress range is not as clear. It appears, at least, that the strength is distinctly lower than the Weathered and slightly less than the Intact strength for this stress range. A discussion of the stress-strain and volume change relations and "before and after test" soil properties will follow to clarify some of the concepts set forth on the Composite Mohr plot (Figure 3.6). Subsequently, theories will be postulated to explain the behavior of the Weathered and Toe materials as opposed to the Intact samples.

3.6 Stress-Strain and Volume Change Relations of the Lake Edmonton Material

In his Oslo Conference General Report, Morgenstern (1967) reviewed in detail the peculiarities and problems associated with stiff clays. He has brought to our attention the facts that stiff clays usually have liquidity indices less than 0.5, have all been moderately to heavily overconsolidated and have a propensity for brittle or strain-softening behavior. Using these characteristics as a guide, the Lake Edmonton material could, therefore, be classified as a stiff clay. This soil has liquidity indices in the range 0.1 to 0.5, is moderately overconsolidated as indicated by the consolidation tests and displays marked strain-softening behavior (Appendix A).

The most notable exception to the strain-softening behavior is the stress-strain curve of Triaxial Test #2, which flattens out rather abruptly after the peak stress is reached. Skempton and Petley (1967) and Marsland (1972) have found similar stress-strain curves to be associated with the strength mobilized along structural dicontinuities such as fissures. This supports the view taken by the author earlier in this subsection.

The consolidated-undrained triaxial tests, on the other hand, all display non-peaked stress-strain curves with, in some cases, very high strains to failure (Tests #2b, #2c and #3b, Table III.4). This fact, which has been alluded to previously, has been explained by Lambe and Whitman (1969):

> "...the tendency toward volume expansion exists out to large strains and consequently the excess pore pressure induced by undrained shear continues to increase to large strains. These decreasing pore pressures imply increasing effective stress, and the stress-strain curve continures to rise out to very large strains."

The results presented by Chandler (1967) on a stiff silty clay also show this effect. In addition, it should be noted at this point, that all the undrained tests have stress paths consistent with overconsolidated behavior (Appendix A).

The volume change-strain characteristics exhibited by both the Intact and Weathered specimens are of much interest. In all cases the samples showed contraction almost to the peak deviator stress, whereupon dilation commenced and continued at various rates which depended on the nature of the failure surface developed. Tests conducted by Bishop et al (1965), Chandler (1967) and Marsland (1972) have demonstrated the same behavior to some degree. The tests undertaken for this investigation have shown that the rougher and more irregular the failure surface, the higher the rate of dilation after peak deviator stress. A comparison of the volume change-strain curves of Triaxial Test #2, which developed a very smooth bi-linear failure plane and Triaxial Test #3 (Plate 6b) attests to this behavior.

The Intact specimens show a decreasing contraction to the peak deviator stress when sheared under lower cell pressures (Table III.3), but no general correlations with the mass structure of the material or strain to failure could be made. The Triaxial Test #4 specimen developed a very smooth failure plane having a film of soil on it in which the silt appeared to be quite close to its liquid limit. It is apparent, therefore, that uhder lower confining stresses more dilation occurs, but the increased dilation is not related to a change in the mode of failure. The Weathered material, on the other hand, does display bi-modal falure characteristics as discussed previously, which can be related to the stress-strain and volume change characteristics of the soil.

The Weathered material yielded results which correspond closely to the work of Marsland (1972) (Figures 3.8 and 3.9) who presented a failure theory for stiff, highly-fissured clays in which the material acts as a granular mass with shear between the lumps below the existing overburden pressure. The resulting Mohr envelope for the low stress range thus has a zero cohesion intercept with a higher ϕ' (Figure 3.9); the effects of overconsolidation are retained only inside the hard lumps. He stated:

> "...At medium stress levels the lumps remained tightly interlocked and a large proportion of the shear plane passed through the intact overconsolidated clay...In the tests at low effective stresses the lumps of clay remained almost intact, having well-defined surfaces covered with thin films of water. When the sheaths were removed from the specimens the slightest touch caused them to fall apart."

This is a particularly appropriate description of the behavior of the Weathered material, as Plate 7 illustrates. The Weathered material shows consistently lower failure strains, a decrease in contraction to the peak deviator stress and higher dilation rates after the peak deviator stress with decreasing cell pressure. These effects are most pronounced when the transition from above to below the overburden stress is made (i.e. Triaxial Test #8 to #9). Triaxial Test #9, which is below the overburden pressure, also has a much less peaked stress-strain curve (Appendix A). The excellent correlation with the work of Marsland, therefore, supports the bi-linear envelope theory for the Weathered soil postulated herein.

As opposed to the drained tests in which the volume change is measured directly, the tendency for volume change in undrained tests can only be sensed through pore pressure response and is discussed in terms of the "A parameter", which relates the pore pressure response to the mobilized shear stress. The A parameter is a highly sensitive indicator of the stress history of the sample, as Lambe and Whitman (1969) point out. It is found to decrease markedly with increasing overconsolidation ratio (Figure 82, p. 118, Bishop and Henkel, 1962).

The Af values calculated for both the Intact and Toe samples decrease with decreasing effective cell pressure (Table III.4), but the effect is much more pronounced in the case of the Toe samples. It is believed that for the Toe specimens, the low Af values under modest confining stresses are an effect of the high dilatant tendency associated with shear around the nugget structure, while the slight decrease of Af noted for the Intact specimens is due simply to shearing under reduced effective consolidation stresses; i.e. the restraint imposed upon the soil grains along the failure plane is not as great and therefore, more dilation may occur. Marsland and Butler (1967) have reported variations in the Af from zero to about 0.65 for effective consolidation pressures of 17 and 210 kPa respectively. This trend for the stiff fissured Barton clay agrees well with the results presented for the Toe samples. The main point which the Af values and the corresponding stress paths (Appendix A) emphasize is that the materials under the normal stress ranges considered are behaving in a dilatant manner slightly before and after the peak deviator stress.

Volume change in the direct shear tests, measured as vertical versus horizontal deformation (plots in Appendix A), did not appear to correlate with the structure of the material tested but rather with the normal stress applied. Most of the tests above approximately 40 kPa contracted to the peak strength while nearly all those below this value dilated. The dilation angles, calculated as the arctangent of the ratio of the vertical to horizontal deformation to the peak shear stress, varied from about 1.5° to 3°. An anomalous behavior, however, occurred in Direct Shear Test #10, which was sheared at the lowest normal stress of 14.5 kPa. The dilation in this test was recorded as about 12°.

The dilation for all tests continued past the point of the peak stress to the end of travel of the shear box. It is obvious, therefore, that a simple relation between peak strength and the dilation angle "i" as presented by Patton (1966) does not hold for the complex soils tested here. Factors believed responsible for the behavior observed for the tests herein are:

1. The plucking or rotation of nuggets in the weathered material, as has been postulated previously, would result in much less dilation than if the asperities were fixed, as is the case for rock.

2. In direct shear tests on undisturbed soil samples, a failure plane is imposed. A non-uniform stress state, plus the effect of restraint as discussed by Morgenstern and Tchalenko (1967) may lead to the formation of Riedel Shears, which are themselves governed by sample composition, texture and structure. In the higher stress ranges, the "riding up" effect along the Riedel Shears tends to be suppressed, which results in contraction, while in the lower stress range the restraint is less, allowing for dilation.

Following the preceding detailed discussion of the strength behavior of the materials tested in this investigation, it is now necessary to postulate theories to account for the strength and structural differences between the three soil types. Physical weathering and stress relief are believed to be the two dominant agents responsible for these differences in behavior; discussion, therefore will center on these phenomena.

3.7 Physical Weathering Processes and Their Effects on Soils

Physical weathering, involving such processes as cycles of wetting and drying and freezing and thawing, is highly dependent on the severity of the climate. Two regions with different climatic conditions, namely the United Kingdom and Western Canada (in particular the Edmonton, Alberta, region), will be discussed in order to demonstrate this point.

The United Kingdom has a West Maritime climate charcterized by cool winters, warm summers and rain throughout the year (Bartholomew, 1977). The East Midlands, where many of the stiff clays crop out, receives about 600 to 700 millimetres of rainfall per year. The annual range in temperature is not great, and there is very little, if any, frost penetration in the winter. Weathering, therefore, would generally proceed under moist conditions.

A limited amount of research has been carried out on the effects of weathering in the United Kingdom, and the general consensus is that it results in an increase in water content with a decrease in strength, approaching the fully softened state. In particular, Chandler (1972) with respect to the Lias Clay, has found:

> "...the effect of weathering being to increase the water content at a given overburden pressure..."

The moisture content-depth profiles which he has presented all show a marked increase in moisture content towards the surface.

Skempton (1970) has drawn an analogy between the "fully softened" state and the normally consolidated strength of the soil through consideration of the critical state of shear deformation as has been discussed in Chapter I. He has defined the effects of weathering on a stiff, overconsolidated clay as a downward shift of the peak envelope, maintaining the same Ø' but with zero cohesion (i.e. the normally consolidated strength). The Weathered material tested for this research project does not show this behavior, but rather shows a slight increase in strength with respect to the Intact material in the higher stress ranges (Figure 3.6)

The climate of Western Canada, and the Edmonton region in particular, contrasts with that of the United Kingdom. Kathol and McPherson (1975) stated:

> "...Edmonton has a climate described as continental, varying between dry and moist subhumid; warm summers and cold winters are typical with moderate precipitation during all seasons."

In particular, the Edmonton region has an average total annual precipitation of about 450 millimetres (Appendix B), of which the winter snowfall comprises slightly greater than 100 millimetres. Large variations from the mean, which may exist for months or even years, are not uncommon and may result in protracted drought or semi-humid conditions (Hamilton, 1968). Below freezing conditions result in substantial frost penetration, ranging from about 1.5 to as high as 2.5 metres, depending on such factors as snow cover, temperature and soil moisture conditions.

One of the most important climatic aspects of the Edmonton region is that the total precipitation, except in rare extremely wet periods, is much below the potential evapotranspiration, resulting in a net desiccation or moisture deficiency in the soil mass (Hamilton, 1963; Hamilton and Tao, 1977). The desiccation, which results in overconsolidation of the soil with the subsequent formation of cracks and fissures (Quigley, 1975; Mitchell, 1976) is usually concentrated in the upper layers of the deposit, resulting in a "drying crust". Moisture contents in these types of deposits decrease toward the surface, with corresponding increases in strength of the material between the cracks and fissures (Lambe and Whitman, 1969, present a number of soil profiles with these characteristics).

The effects of desiccation are believed to have caused much of the overconsolidation apparent in both the Intact and Weathered samples tested in research of this thesis. The Weathered 10.2 centimetre diameter triaxial samples were found to have a slightly lower average moisture content than the Intact specimens (35.6% compared to 37.3%; Table III.6) and the borehole log for piezometer installation A (Figure 2.3) revealed moisture contents increasing from 21.4% at a depth of 0.8 metres to 39.2% at a depth of 3.3 metres (Thomson, 1978). Futhermore, pre-construction drilling reports for the site indicate moisture contents increasing with depth, from approximately 29 to 34% at a depth of two metres to about 38 to 40% at a depth of eight metres (City of Edmonton Report, 1967). The Weathered specimens, being in a zone of more intense desiccation, are therefore more overconsolidated and fractured than the Intact samples. Subsequent wetting, due to the downward percolation of rainwater or spring runoff, would result in some swelling, but the soil would not swell back to the original void ratio present before desiccation. Significant softening around the fissures or joints was not apparent in either the Weathered lab samples or in the field.

Frost action can also cause overconsolidation. Mackay (1974), in suggesting a theory for the formation of reticulate ice vein networks, has found that the clay blocks or nuggets between the ice veins are often overconsolidated when thawed. He attributes this phenomenon to the formation of the ice veins behind the aggrading zero degree isotherm in an essentially closed system. He stated:

> "...If ice veins grew well above the lower permafrost surface, some water would be withdrawn from the adjacent clay, both above and below the enlarging ice veins...Shrinkage of the clay would accompany the loss of water; ice veins would tend to grow in the shrinkage

> cracks so produced, and the clay would gradually become consolidated..."

McRoberts and Nixon (1975), in a discussion of the Mackay (1974) paper, have postulated a theory in which cracking occurs in the unfrozen soil in advance of the freezing front. The cracking is believed to be a result of the suction forces which create non-uniform volume changes (consolidation) in the soil mass. Water is subsequently drawn from the immediately adjacent soil into the cracks and frozen as the zero degree isotherm passes.

With each subsequent cycle of freezing and thawing, it is believed that the advance of the freezing front will cause ice lensing to occur in the pre-existing cracks, drawing water from the adjacent blocks or nuggets and, therefore, increasing the overconsolidation. Upon each thaw cycle, some swelling will occur , but there is a noticeable hysteresis (i.e. the original void ratio before the freezing cycle will never be attained). The effects of cycles of freeze-thaw would, therefore, be:

1. An increase in the density of the intact nuggets.
2. A decrease in void ratio.
3. Degree of saturation close to 100%.

The Weathered soil tested herein has a slightly higher density than the Intact material (18.2 kn/m.^{3} compared with 17.9 kn/m.^{3}), slightly lower void ratio (1.00 compared with 1.05) and a saturation close to 100% (97.4%), based on values averaged from the 10.2 centimetre diameter triaxial specimens (Table III.6).

The Toe samples, on the other hand, display the structural effects of freezing (i.e. nuggety structure) but not the other properties such as densification and lower void ratios. The probable reason for this is that the immediate swelling due to the excavation of about five metres of overlying soil has had a greater effect than subsequent consolidation effects of freezing. The soil has only been exposed to freezing since construction of the cut, and therefore, has not experienced the number of cycles that the Weathered material has undergone.

An example of the destructive effects of frost penetration on a soil structure is the case of a starter dyke at the Oil Sands Site at Fort MacMurray, Alberta (Watts, personal communication). A reticulate ice vein network, with the veins as thick as five millimetres was noted to have formed to a depth of about 2.5 metres in the unprotected, impervious core after just one winter. The network was found to be much more intense in the glaciolacustrine clay sections (containing about 54% clay sizes) of the core than in the sandy till sections (containing about 20% clay sizes). The glaciolacustrine clay was placed at a moisture content slightly higher than the till. It could be concluded, therefore, that the frequency of ice lense formation and the associated structural breakdown is directly related to the moisture content and activity of the soil, which is partially reflected by the percent clay sizes.

3.8 Conclusions of the Test Results

In summary, the effects of sample variability, heterogeneity and complexity, both in composition and structure should be stressed. There is much scatter in the test results, as other workers who have dealt with overconsolidated, stiff fissured clays have found (e.g. de Beer, 1967). Anomalies in the results, however, have been rationally explained and the parameters defined on the Composite Mohr Plot (Figure 3.6) are believed to best describe the strength characteristics of the various materials tested.

TABLE III.1

SUMMARY OF INDEX TESTS ON SELECTED SAMPLES

Material	Wl	Wp	Ip	% Silt	% Clay	Specific Gravity
dark gray silty clay	98	37	61	23	77	2.76
buff clayey silt	44	23	21	75	25	2.72

NOTES: Tests were performed in accordance with procedures outlined in ASTM (1977).

X-RAY DIFFRACTION RESULTS

Material	% Clay Minerals Present				
	Montmorillonite	Mica	Kaolinite	Inter Stratified Mica-Montmorillonite	Quartz
dark gray silty clay	60	15	15	7	3
buff clayey silt	70	10	10	7	3

Notes: Tests were performed on the clay size fraction of each sample.
Results can be considered accurate to within ±10%.
Tests were undertaken by the Department of Soil Science, University of Alberta.

TABLE III.2

CONSOLIDATION TEST RESULTS

Pressure (kPa)	e	Cv (Cs) (cm^2/sec x10^{-4})	K (cm/sec x10^{-8})
10.7	1.055	9.5	3.1
53.6	1.026	2.16	0.76
96.6	0.993	3.3	1.15
300.0	0.921	2.3	0.41
53.6	0.973	1.7	0.21
96.6	0.961	2.0	0.25
300.0	0.917	2.17	0.25
600.0	0.845	1.1	0.13
10.7	0.996	0.6	-

Notes: Initial moisture content = 36.4 %.
Initial Saturation = 94.1 %.
Average Cv for initial loading to 96.6 KPa = 5.0 x 10^{-4} cm^2/sec.
Average Cs = 1.11 x 10^{-4} cm^2/sec.

Cv determined from log time versus settlement plots.

TABLE III.3

CONSOLIDATED-DRAINED TRIAXIAL TEST RESULTS - 10.2 cm. AND 3.8 cm. DIAMETER SAMPLES

Test #	Effective Consolidation Pressure (kPa)	Maximum Deviator (kPa)	Corrected* Maximum Deviator (kPa)	$\sigma_1'_f$	$\sigma_3'_f$	pf	qf	$\varepsilon_f\%^+$	$\Delta V/V_f\%$
Intact									
1	82.7	137.1	132.3	215.0	82.7	148.85	66.15	3.8	-1.08
2	62.1	81.5	76.7	138.8	62.1	100.45	38.35	3.7	-0.89
3	41.4	125.0	120.2	161.6	41.4	101.5	60.1	4.7	-0.77
4	20.7	65.9	61.1	81.8	20.7	51.25	30.55	4.5	-0.41
5	51.7	108.1	103.3	155.0	51.7	103.35	51.65	6.4	-1.10
5a	34.5	116.9	103.1	137.6	34.5	86.05	51.55	3.9	-0.89
5b	69.0	132.8	119.0	138.0	69.0	128.5	59.5	4.0	-1.39
5c	103.4	178.4	164.6	268.0	103.4	185.7	82.3	4.6	-1.61
Weathered									
6	82.7	176.3	171.5	254.2	82.7	168.45	85.75	4.3	-1.14
7	62.1	144.8	140.0	202.1	62.1	132.1	70.0	4.1	-1.26
8	41.4	109.2	104.4	145.8	41.4	93.6	52.2	3.9	-1.26
9	20.7	68.8	64.0	84.7	20.7	52.7	32.0	3.1	-0.5[illegible]

Notes: A backpressure of 206.8 KPa was used in all tests.

* Deviator stress correction for 10.2 cm. Samples is 4.8 KPa and for 3.8 cm. Samples is 13.6 KPa.

Tests 5a,5b,5c are 3.8 cm. diameter Samples; the rest are 10.2 cm. diameter Samples.

+ Strain to failure is only approximate; initial strains for load cap readjustment are discounted.

TABLE III.4

CONSOLIDATED-UNDRAINED TRIAXIAL TEST RESULTS - 3.8 cm. DIAMETER SAMPLES

Test*	Effective Consolidation Pressure (kPa)	Maximum Deviator (kPa)	Corrected Maximum Deviator (kPa)	U (kPa)	$\sigma_1'_f$	$\sigma_3'_f$	Pf	Qf	Af	ε_f%
Intact										
1B	100.0	110.6	96.8	38.6	158.2	61.4	109.8	48.4	0.35	3.7
1C	60.0	65.2	51.4	22.8	88.6	37.2	62.9	25.7	0.35	4.2
1D	30.0	64.6	50.8	11.55	69.25	18.45	43.85	25.4	0.16	2.26
TOE										
2A	60.0	82.5	68.7	26.1	102.6	33.9	68.25	34.35	0.32	5.3
2B	40.0	94.0	80.2	12.8	107.4	27.2	67.3	40.1	0.14	7.8
2C	20.0	46.1	32.3	-1.2	53.5	21.2	37.35	16.15	-0.03	7.5
3A	70.0	86.9	73.1	38.0	105.1	32.0	68.55	36.55	0.44	3.6
3B	50.0	86.6	72.8	12.4	110.4	37.6	74.0	36.4	0.14	7.5
3C	30.0	65.46	51.66	4.8	76.86	25.2	51.03	25.83	0.07	3.5

Notes: A backpressure of 200 kPa was used in all tests.
Effective consolidation pressure = cell pressure - back pressure.
Deviator stress correction for 3.8 cm. Samples = 13.8 kPa.
The tests were undertaken by graduate students of the Civil Engineering Department as a part of their Post-graduate Laboratory course.

TABLE III.5

DIRECT SHEAR TEST RESULTS

Test#	Normal Stress (kPa)	Peak Shear Strength (kPa)	Residual* Shear Strength (kPa)	Horizontal Displacement To Peak (cm.)	ΔH/H % at Peak
Intact					
1	82.7	57.2	30.0	0.072	-0.12
1A	102.9	53.95	29.0	0.332	-0.58
1B	31.3	27.3	8.0	0.22	0.18
1C	19.0	22.85	-	0.23	0.11
2	62.1	50.9	-	0.19	0.045
3	41.4	23.0	-	0.083	0.10
4	20.7	23.6	-	0.068	0.11
Weathered					
5	82.7	50.3	18.0	0.15	-0.20
6	62.1	46.6	17.0	0.16	0.022
7	41.4	37.55	-	0.255	0.56
8	20.7	19.2	-	0.197	0.15
9	31.0	29.2	11.0	0.175	0.35
10	14.5	23.5	7.0	0.087	0.78
Toe					
2A	60.0	43.3	22.0	0.17	-0.081
2B	40.0	39.05	11.0	0.056	-0.76
2C	20.0	17.8	-	0.14	-0.10
3A	70.0	39.4	-	0.22	-0.88
3B	50.0	21.3	-	0.017	0.002
3C	30.0	16.90	-	0.065	0.00

NOTES: * As determined from plots; so only approximate.
The Toe Sample tests were undertaken by graduate students of the Civil Engineering Department as a part of their Post-Graduate Laboratory Course.

TABLE III.6

CONSOLIDATED - DRAINED TRIAXIAL TESTS - 10.2 cm. AND 3.8 cm. DIAMETER SAMPLES

SOIL PROPERTIES : BEGINNING OF TEST : END OF TEST

Test #	Wet Density (KN/M³)	Saturated Density (KN/M³)	Average Moisture Content (%)	Void Ratio	Degree Of Saturation (%)	Moisture Content -Failure Plane	Moisture Content -Sample	Dip Angle Of Failure Plane
Intact								
1	17.94	18.12	37.00	1.05	96.5	38.64	38.00	70° to 50° - Dual planes
2	17.58	17.58	42.75	1.17	100.0	45.09	38.95	40° to 60° - Bi-angular
3	18.06	18.37	34.37	0.99	94.1	35.10	35.79	about 53° at top
4	18.26	18.60	31.93	0.94	93.3	36.93	36.03	about 57° - Uniform
5	17.74	17.74	40.62	1.12	99.4	40.25	39.74	40° to 50° - Bi-angular
5a	17.74	17.89	39.63	1.11	97.8	43.54	40.51	50° to 55° - Uniform
5b	17.54	17.63	42.33	1.18	98.6	43.58	38.96	40° to 60° - Bi-angular
5c	17.54	18.01	35.66	1.08	90.5	41.90	40.00	45° to 53° - Conjugate
Weathered								
6	18.47	18.47	35.26	0.97	99.6	35.34	33.03	about 60° - Stepped
7	17.94	18.24	35.14	1.02	94.0	35.90	34.11	37° to 57° - Stepped
8	18.33	18.37	35.49	0.99	98.3	35.44	33.71	52° to 60° - Uniform
9	18.13	18.24	36.34	1.02	97.6	37.75	36.62	55° to 77° - Irregular

Notes: Tests 5a, 5b and 5c are 3.8 cm. Diameter tests.
G was assumed 2.74 in all calculations.

TABLE III.7

CONSOLIDATED - UNDRAINED TRIAXIAL TESTS - 3.8 cm. DIAMETER SAMPLES

SOIL PROPERTIES : BEGINNING OF TEST : END OF TEST

Test#	Wet Density (KN/M³)	Saturated Density (KN/M³)	Average Moisture Content (%)	Void Ratio	Degree Of Saturation (%)	Moisture Content-Failure Plane	Moisture Content-Sample	Dip Angle of Failure Plane
Intact								
1B	17.75	18.04	38.9	1.09	97.0	41.6	-	-
1C	17.75	18.24	36.5	1.05	94.0	41.5	41.8	45° and 60° - Dual
1D	18.24	18.24	39.1	1.03	100.0	40.5	38.9	48°
Toe								
2A	16.84	16.87	49.4	1.385	97.7	-	46.3	44°
2B	17.52	17.52	48.0	1.271	100.0	47.4	48.4	36°
2C	17.38	17.38	46.9	1.272	100.0	45.7	48.9	57°
3A	17.65	17.65	47.4	1.25	100.0	41.2	40.7	55°
3B	17.26	17.42	44.2	1.24	97.7	46.6	42.8	46°
3C	17.06	17.46	41.9	1.23	93.3	45.8	47.7	62°

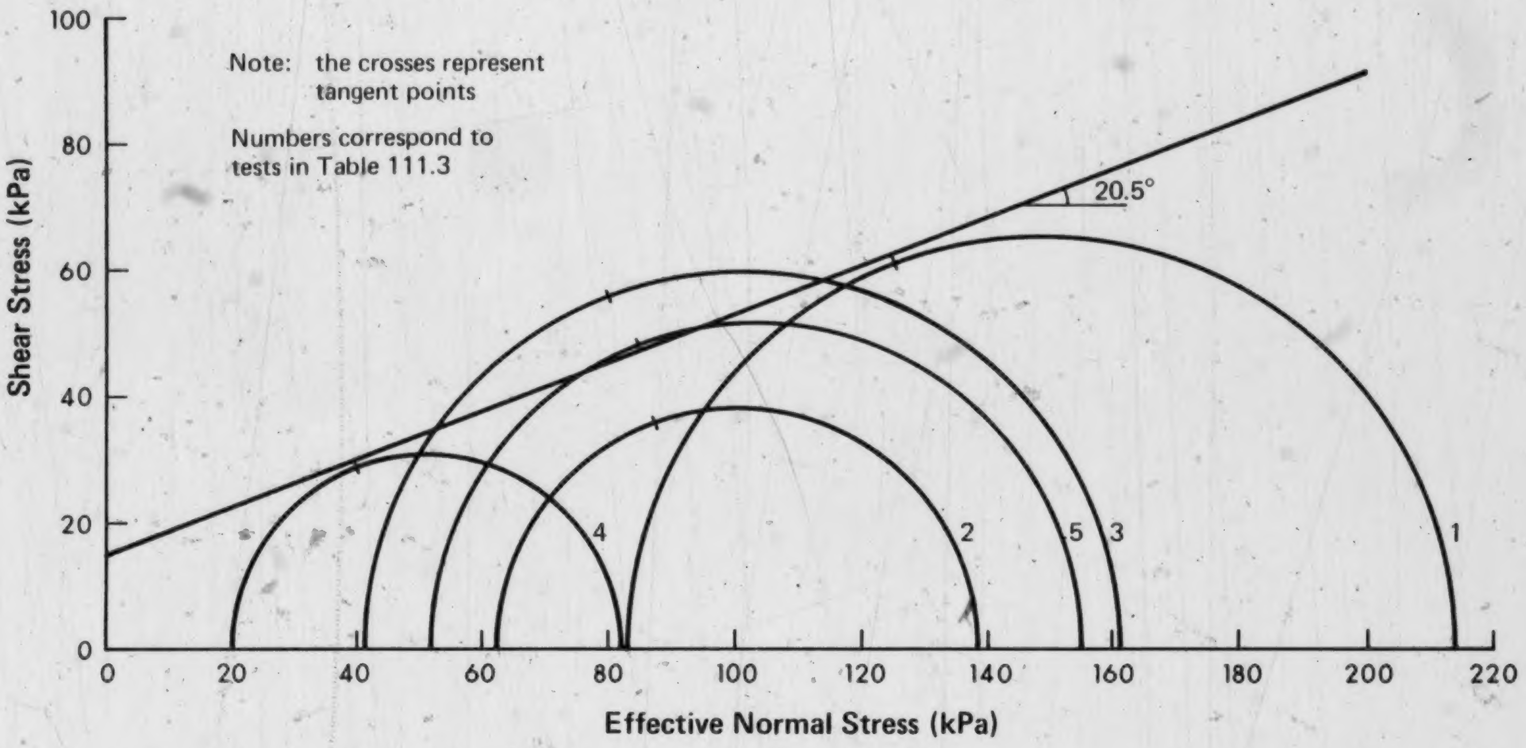
Note: G was assumed 2.74 in all calculations.

TABLE III.8

DIRECT SHEAR TESTS

SOIL PROPERTIES : BEGINNING OF TEST : END OF TEST

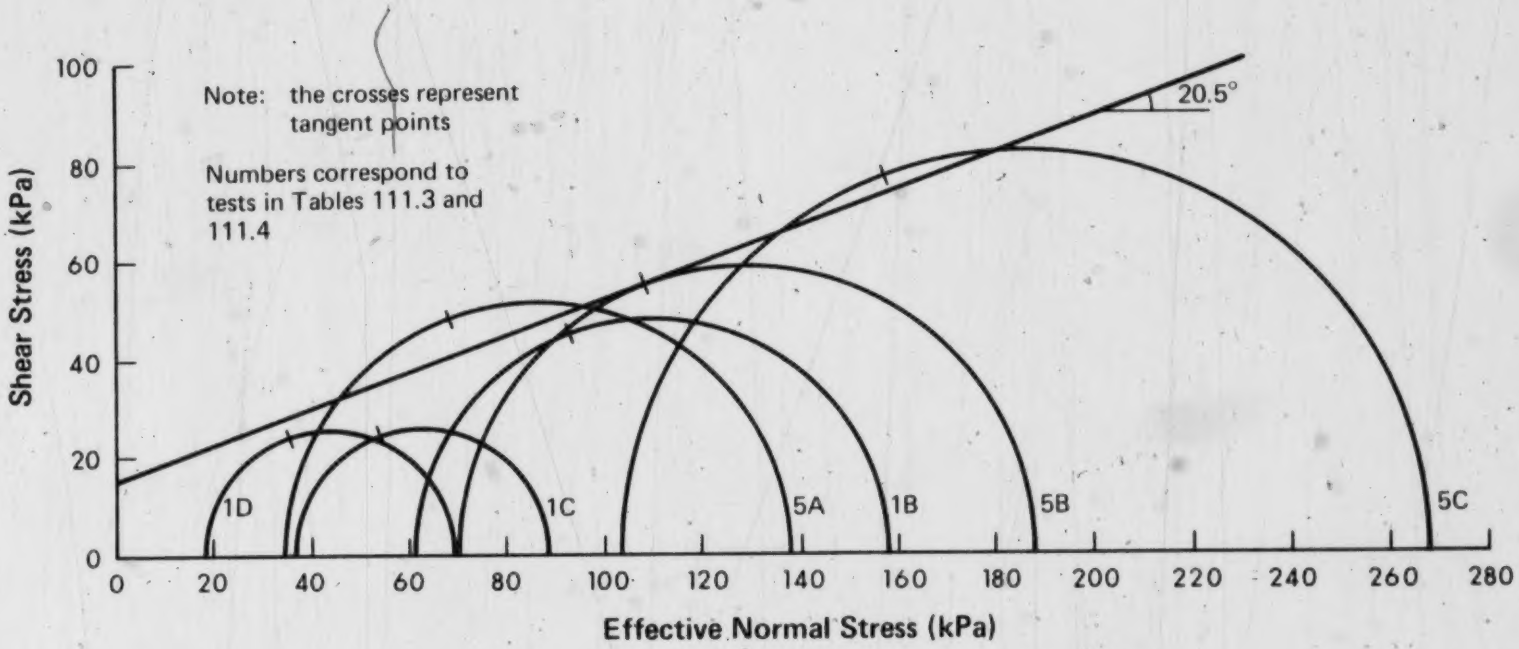
Test#	Wet Density (KN/M³)	Saturated Density (KN/M³)	Average Moisture Content (%)	Void Ratio	Degree Of Saturation (%)	Moisture Content-Failure Plane	Moisture Content-Sample
Intact							
1	18.44	18.60	33.41	0.94	97.4	37.98	34.70
1A	16.74	17.36	41.0	1.26	89.2	40.3	37.70
1B	17.26	18.25	32.0	1.02	85.0	45.0	43.05
1C	17.75	17.98	36.6	1.04	95.0	47.2	39.3
2	18.14	18.60	31.56	0.94	92.0	38.85	35.10
3	16.87	17.53	39.0	1.21	87.5	56.95	47.38
4	16.77	17.16	45.16	1.32	93.7	59.78	49.91
Weathered							
5	17.95	18.30	34.82	1.01	94.5	42.53	37.94
6	18.14	18.34	34.94	1.00	94.8	41.13	36.37
7	17.75	18.17	34.60	1.04	91.2	50.00	42.11
8	17.55	18.17	33.10	1.04	87.21	45.72	38.58
9	18.24	18.51	32.40	0.96	92.5	41.13	34.99
10	17.75	18.09	35.98	1.06	93.0	44.92	39.22
Toe							
2A	17.67	17.99	39.10	1.11	96.5	44.7	44.6
2B	17.40	17.84	39.60	1.16	93.5	44.7	40.4
2C	17.37	17.65	42.70	1.21	96.7	38.5	37.6
3A	16.97	17.60	38.2	1.19	88.0	-	-
3B	-	-	-	-	-	-	-
3C	-	-	-	-	-	-	-



Note: the crosses represent tangent points

Numbers correspond to tests in Table 111.3

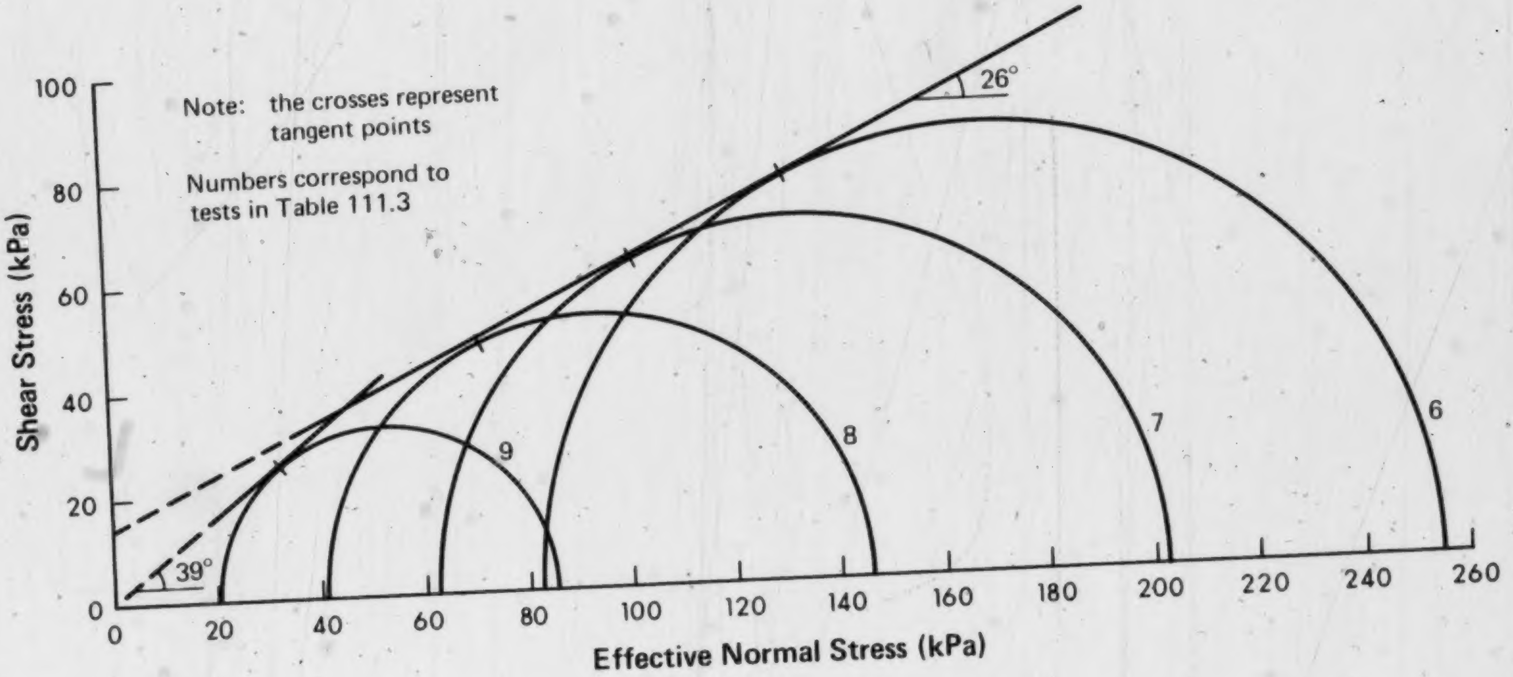
Figure 3.1 Mohr Envelope for 10.2 cm Diameter Intact Drained Triaxial Tests



Note: the crosses represent tangent points

Numbers correspond to tests in Tables 111.3 and 111.4

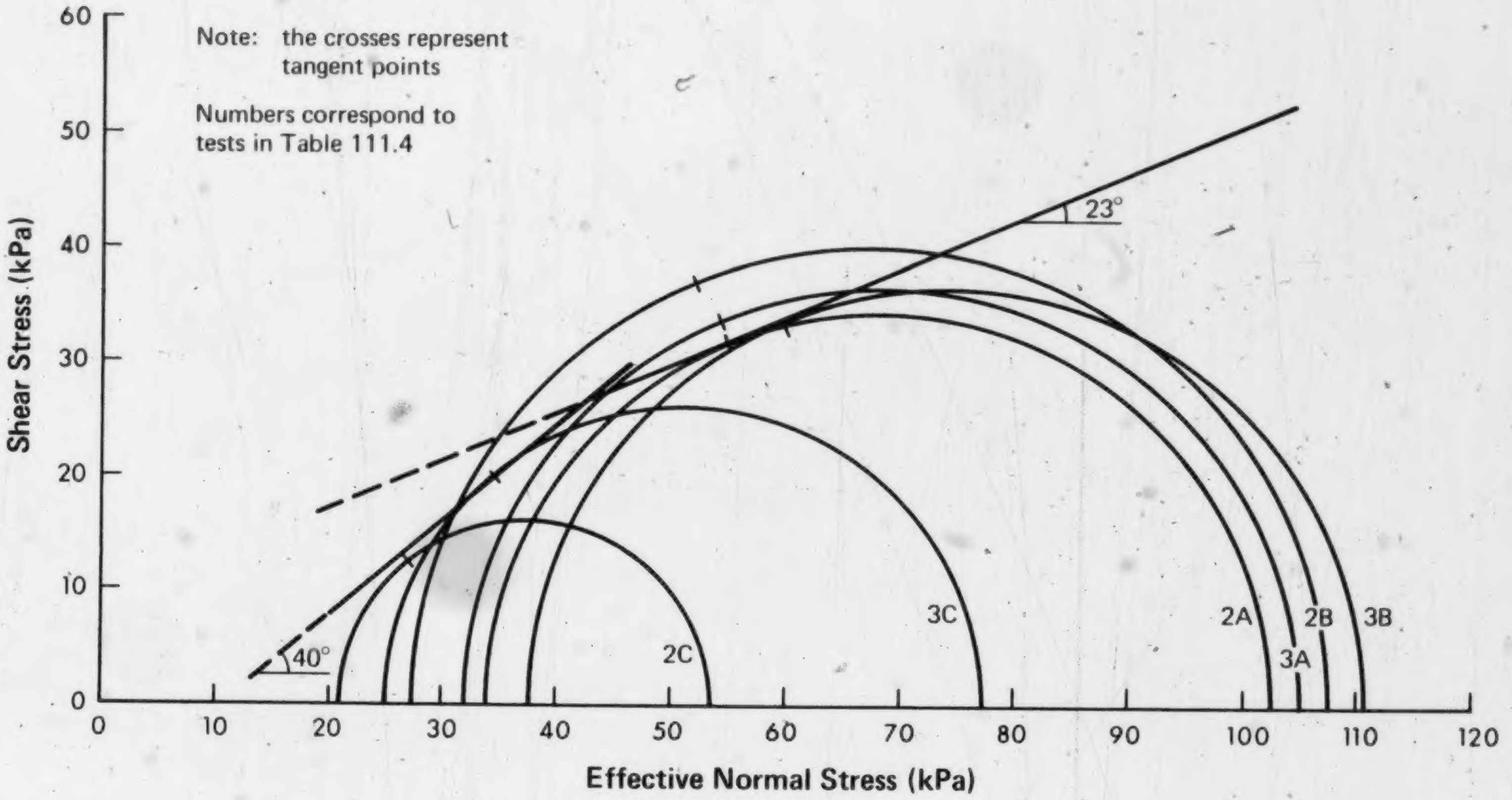
Figure 3.2 Mohr Envelope for 3.8 cm Diameter Intact Drained and Undrained Triaxial Tests



Note: the crosses represent tangent points

Numbers correspond to tests in Table 111.3

Figure 3.3 Mohr Envelope for 10.2 cm Diameter Weathered Drained Triaxial Tests



Note: the crosses represent tangent points

Numbers correspond to tests in Table 111.4

Figure 3.4 Mohr Envelope for 3.8 cm Diameter Toe Undrained Triaxial Tests

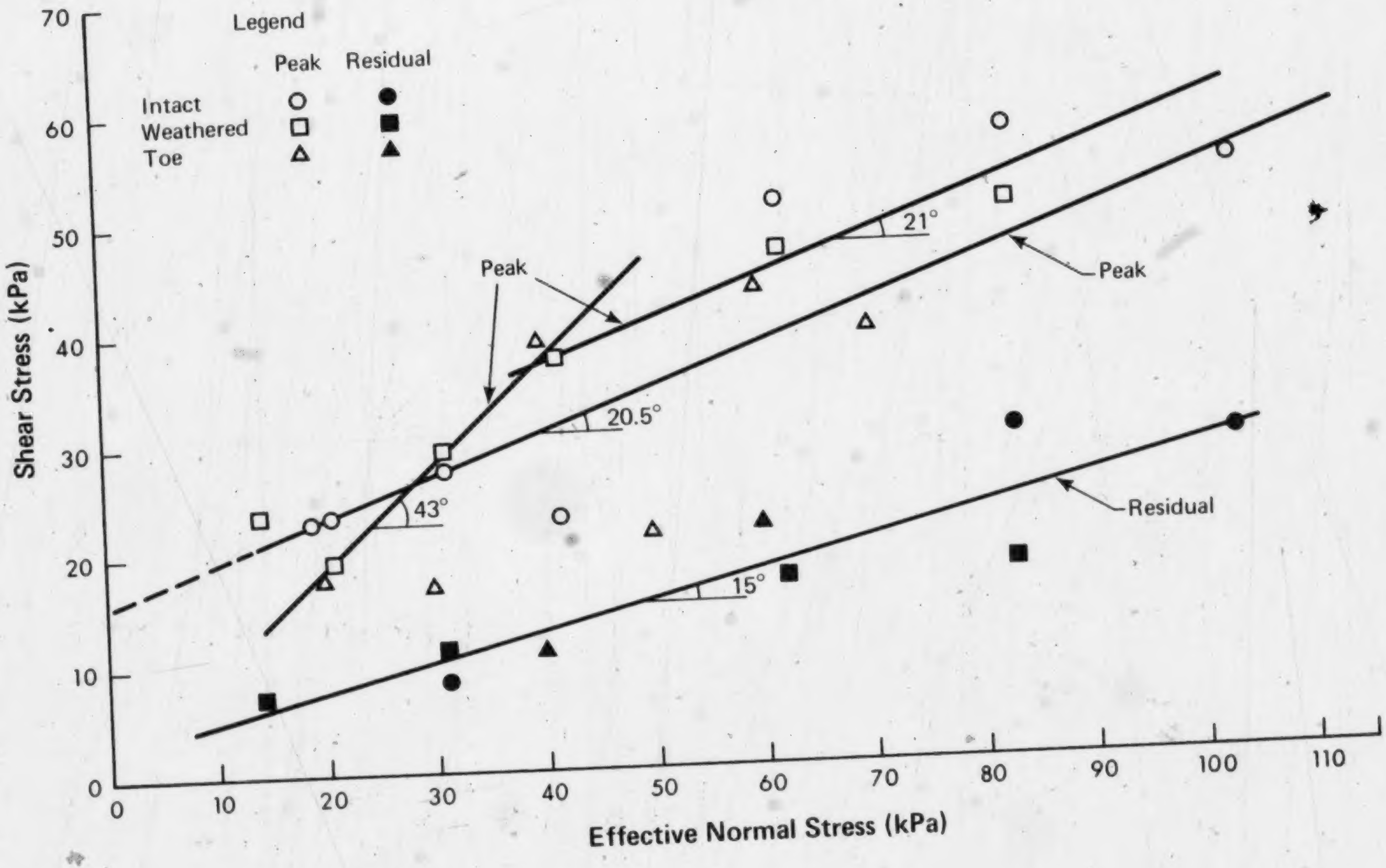
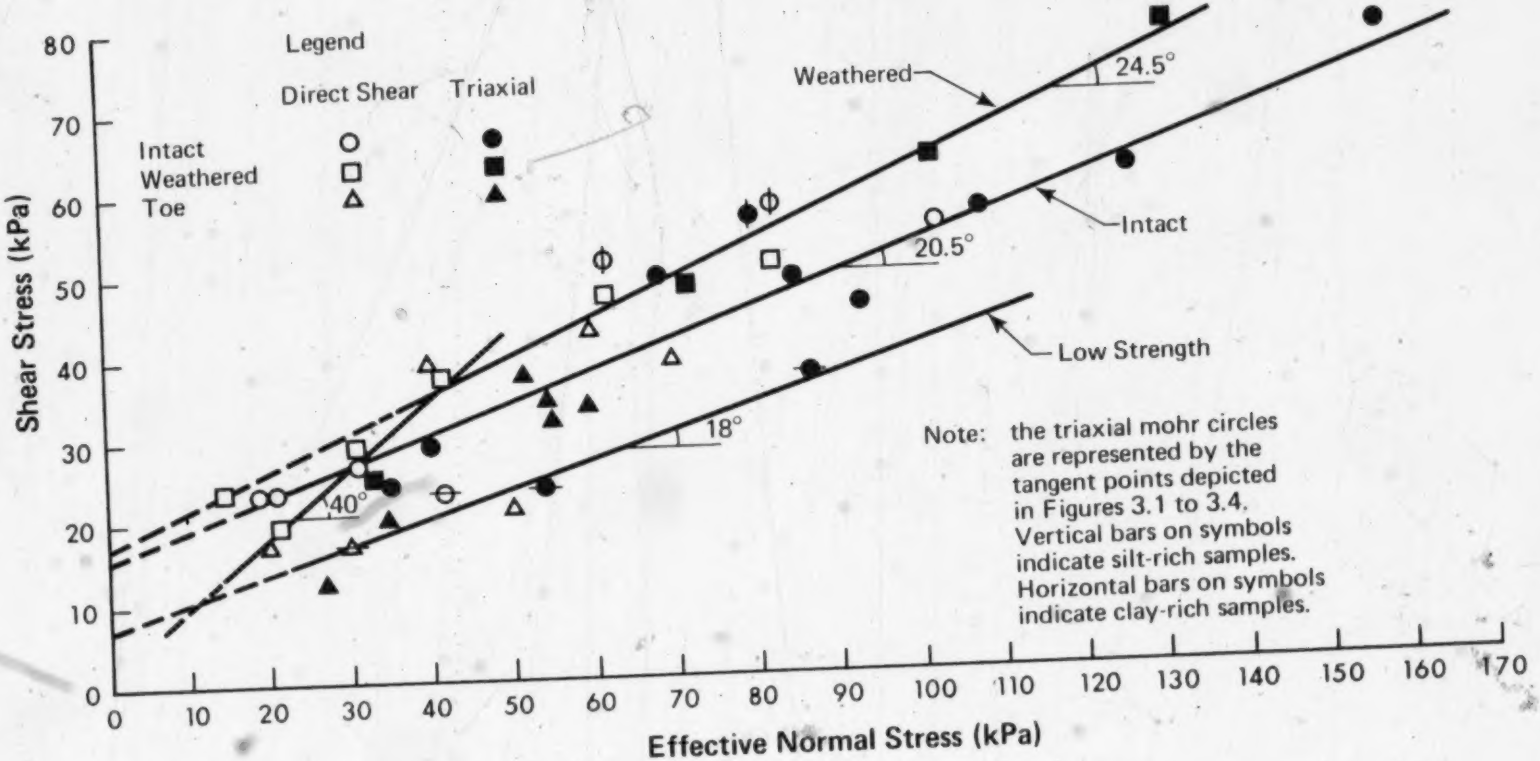
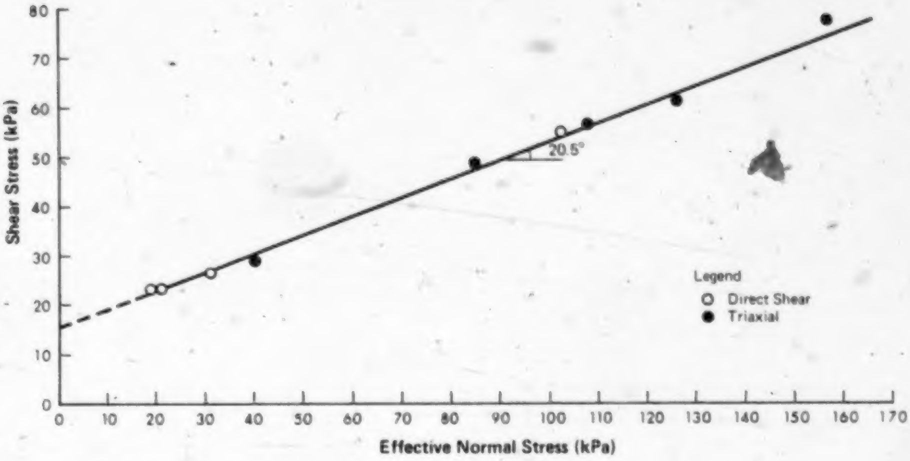


Figure 3.5 Mohr Envelopes for Direct Shear Tests

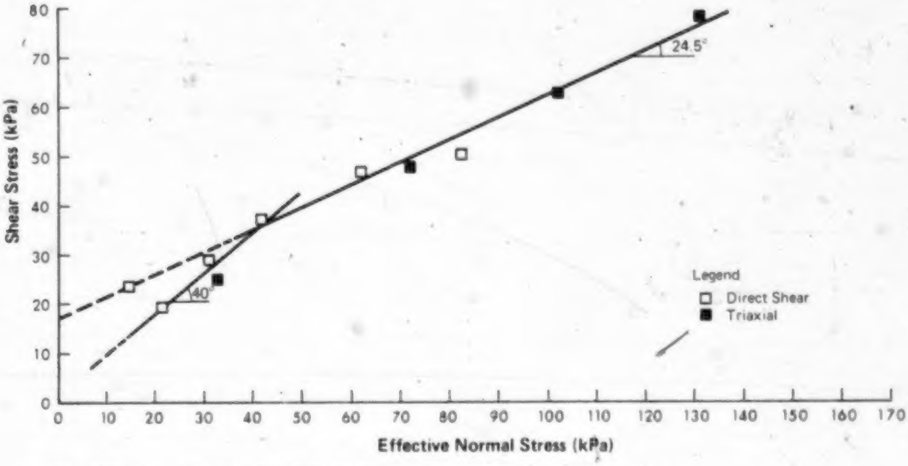


Note: the triaxial mohr circles are represented by the tangent points depicted in Figures 3.1 to 3.4. Vertical bars on symbols indicate silt-rich samples. Horizontal bars on symbols indicate clay-rich samples.

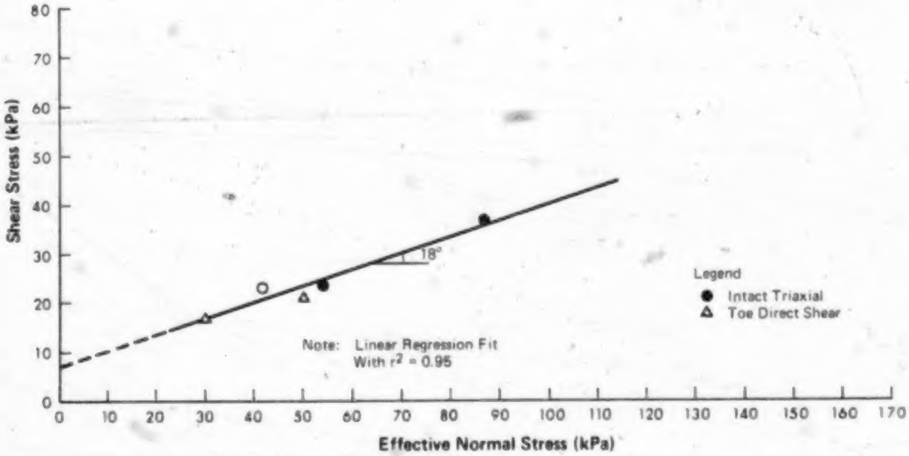
Figure 3.6 Composite Mohr Plot Representing Peak Strength of Direct Shear and Triaxal Data



a) Peak Strength Envelope for Intact Samples



b) Peak Strength Envelope for Weathered Samples



c) Peak Strength Envelope for Low Strength Intact and Toe Samples

Figure 3.7 Composite Mohr Diagrams for Peak Strength of Intact, Weathered and Low Strength Samples

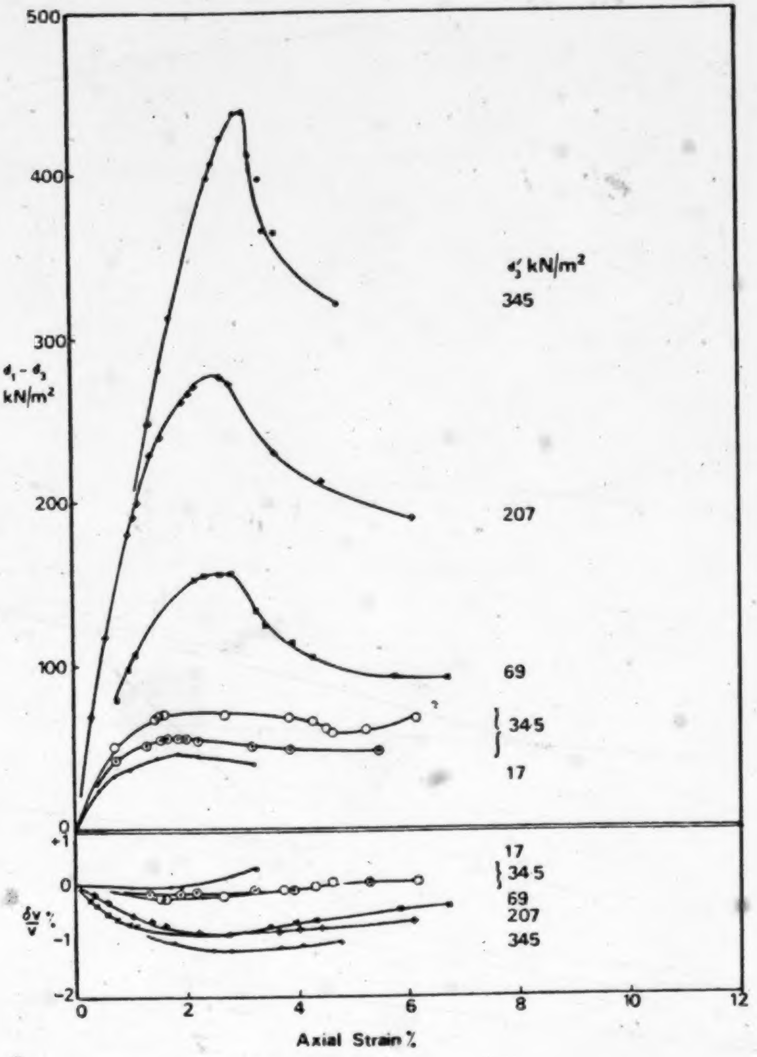


Figure 3.8 Stress-Strain Curves for Drained Triaxial Tests on 98 mm Diameter Specimens of London Clay from Wraysbury (After Marsland,1972)

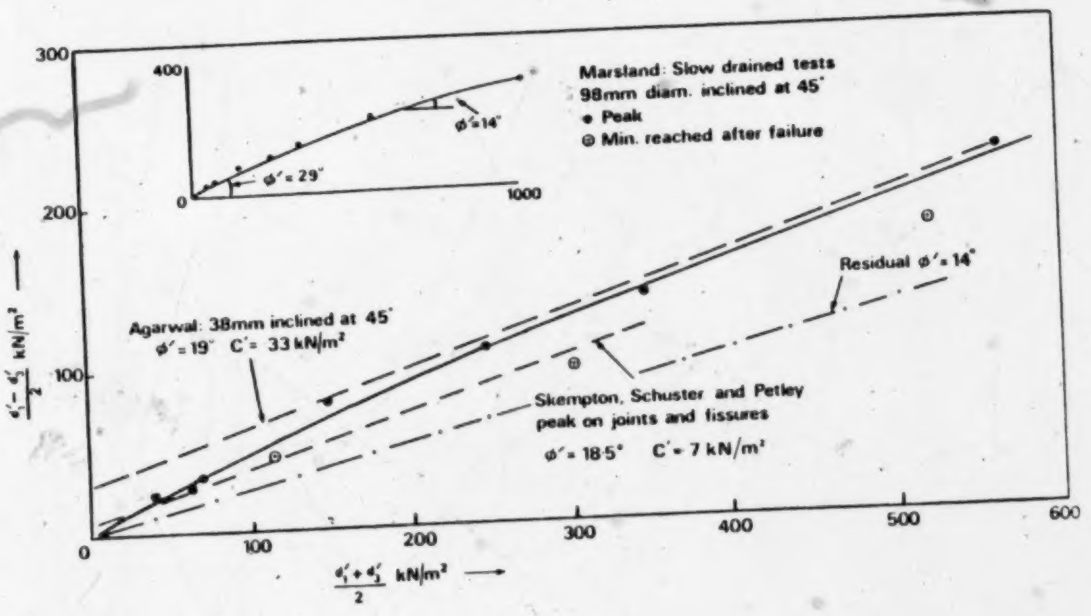
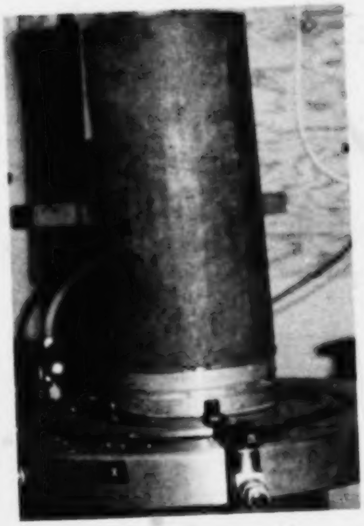
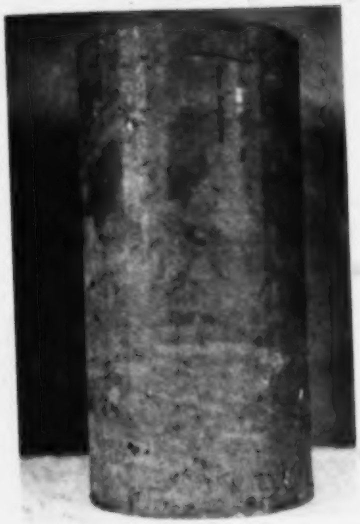


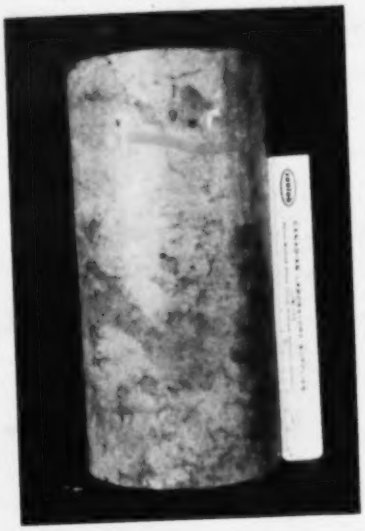
Figure 3.9 Strength of Highly Fissured London Clay from Wraysbury in Terms of Effective Stresses (After Marsland, 1972).



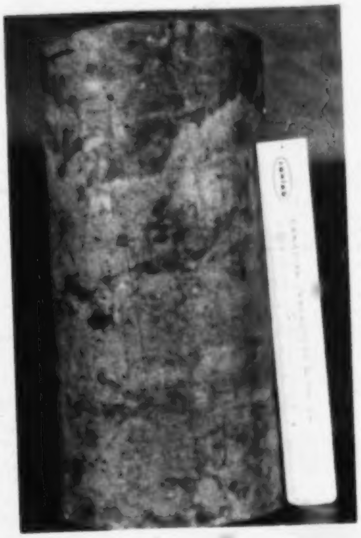
5a Sample T-44(A);
Test #4.



5b sample T-42(A);
Test #5.

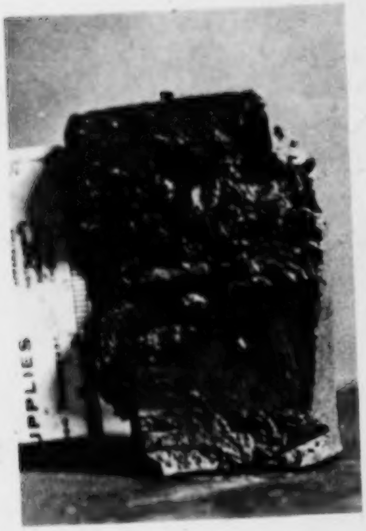


5c Sample T-38(B);
Test #8.

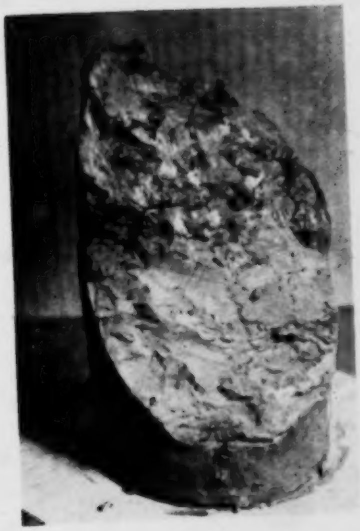


5d Sample T-39(A);
Test #9.

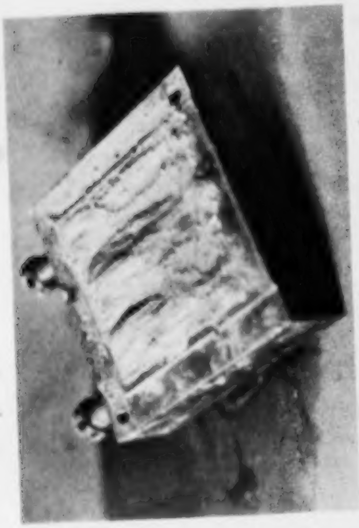
Plate 5 10.2 cm. Diameter Triaxial Test Samples.



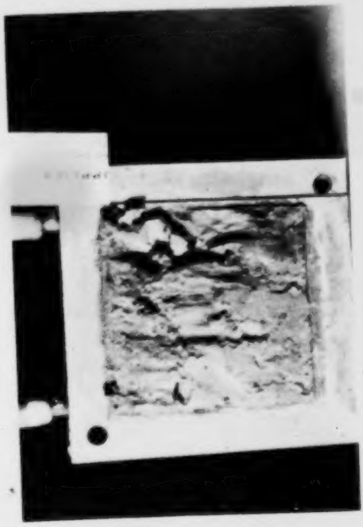
6a Section of triaxial sample T-29(B); Test #3C.



6b Failure plane of triaxial sample T-43(C); Test #3.

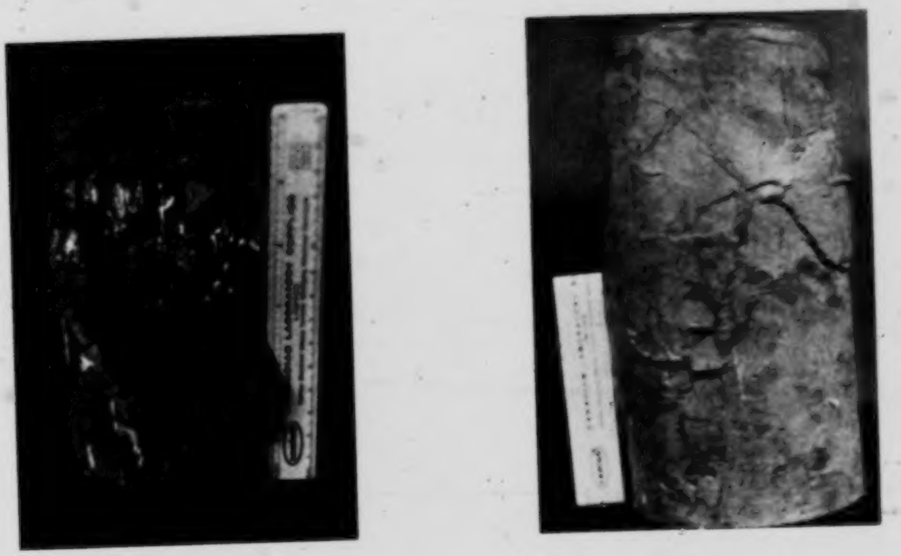


6c Failure plane of direct shear sample T-34(A); Test #5.



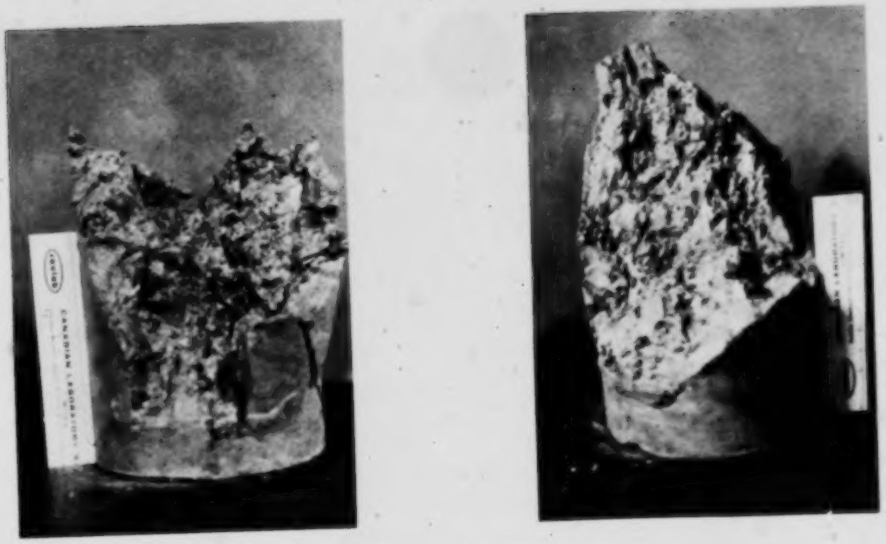
6d failure plane of direct shear sample T-39(A); Test #8.

Plate 6 Triaxial And Direct Shear Samples After Testing.



7a Failure plane of sample T-38(A);Test #6.

7b Sample T-39(A); Test #9.



7c Bottom section of sample T-39(A).

7d Top section of sample T-39(A).

Plate 7 10.2 cm. Diameter Weathered Triaxial Samples After Testing.

Chapter IV

Stability Analysis and Discussion

4.1 Stability Analysis

4.1.1 General

A series of landslides have occurred along the Whitemud Freeway between 156th and 159th Streets following the completion of construction of the cut in 1969 (Chapter I). The slides, characterized by non-circular slip surfaces, were first-time, delayed (or progressive) failures as discussed in Chapter I. The 1973 slide had been investigated in the fall of 1973 and the results of that investigation are presented in Chapter II. This slide will be re-analyzed in detail in this section. The purpose of the stability analysis undertaken for this thesis is twofold:

1. An analysis of the landslide using strength parameters c' and ø', determined by laboratory testing and reasonable field conditions yielded acceptable factors of safety. Thus, plausible mechanisms of failure may be proposed.
2. The delayed aspect of the failures may be explored by varying the strength parameters and groundwater conditions in the sliding mass which would be consistent with the effects of weathering, and noting

the subsequent changes in the factor of safety with time.

A discussion of the method of stability analysis and the choice of pore pressures and soil parameters used in the study follows. The results of the stability analyses performed and a discussion of the proposed mechanisms of failure are presented in the following subsections.

4.1.2 Method of Analysis and Selection of Pore Pressure and Strength Parameters

The Morgenstern-Price (1965) stability analysis of general slip surfaces has been utilized for the study of the 1973 slide. A complete description of the development and applications of the method may be found in Morgenstern and Price (1965) and "The Analysis and Design of Rock Slopes" (1971).

The profile used for the analysis is depicted in Figure 4.1. The configuration of the ground surface and failure plane was determined using the survey, borehole and toe trench data presented in Chapter II (Figures 2.4 to 2.7). Two soil types have been specified on the diagram, with the division approximately 2.3 metres below the preslide ground surface. The upper layer represents the zone of intense physical weathering after the cut had been excavated. The

Lake Edmonton Clay/Till boundary is at a depth of about 7.9 metres below the crest of the slope. Since this layer does not enter into the analysis, it was arbitrarily assigned the Soil #2 parameters. A discussion of the factors involved and assumptions made in determining suitable piezometric levels for the analysis will be discussed in the following paragraphs.

Bishop and Bjerrum (1960) illustrated the theoretical pore pressure responses and changes in the factor of safety during and after the excacvation of a slope in clay. They point out that upon rapid excavation, the pore pressures could be expected to decrease in an undrained mode (i.e. without volume change) due to the unloading. With time, the initial negative excess pore pressures will dissipate and eventually reach a steady state or equilibrium condition, therefore causing a time-dependent decrease in stability.

Eigenbrod (1972, 1975) has analyzed the process of pore pressure equaliztion numerically using finite element and finite difference techniques. The slope was assumed to be a homogeneous, isotropic, elastic material under plane strain conditions. The effects of the rotation of pricipal stresses were neglected. Utilizing the total stress changes computed from a finite element program and an assumed in-situ pore pressure parameter A, the pore pressures immediately after excavation were calculated. The initial negative excess pore pressures were computed as the difference between the initial values derived from the finite element analysis and final pore pressures assumed from a long-term, stabilized groundwater condition.

The time for equalization of the exces pore pressures may susequently be calculated using the two-dimensional finite difference consolidation program developed by Koppula (1970), assuming that swelling and consolidation follow similar theories. Eigenbrod (1972) performed the analyses for the general case of a 30 metre high slope with an inclination of two horizontal to one vertical. Full equalization was attained for a time factor $T=0.33$. The analysis was found to be relatively insensitive to the assumed values of K and A, but was highly dependent on the coefficient of swelling, Cs . With the time factor for full pore pressure equalization T set equal to 0.33, Eigenbrod (1972) investigated the influence of slope height H and the coefficient of swelling Cs on the time for dissipation t of the excess pore pressure. The results were plotted and one of his graphs is presented in Figure 4.2.

For the case of the slope failures in the Lake Edmonton sediments, a time for pore pressure equalization may be estimated directly from Figure 4.2, as the original slopes were cut very close to two horizontal to one vertical (Figure 4.1) and the values assumed for Ko and A are reasonable. Therefore, using the average coefficient of swelling of 1.2 X 10^{-4} cm^2/sec. (4.1 ft^2/yr.) as determined in the laboratory tests (Chapter III) for a slope 7.3 metres (22 feet) high yields an equalization time of about 40 years. This value, however, corresponds to a homogeneous soil with a uniform coefficient of swelling of 4.1 $ft^2/yr.$, and as discussed in Capter III, the in-situ coefficient of swelling for the heterogeneous, discontinuous Lake Edmonton deposit is believed to be much greater than the laboratory value. Furthermore, Rowe (1972) and McGowan and Radwan (1975) have shown that this secondary or mass permeability is stress-dependent, with permeabilities at least a factor of ten higher below the overburden pressure than above it. This may be explained by the fissures and discontinuities opening up at pressures less than the overburden while remaining closed at higher stress levels (McGowan and Radwan, 1975).

Therefore, it is postulated that equalization has taken place in the Whitemud Freeway cut well within the times to failure of the slopes for the following reasons:

1. The stress relief due to the excavation has resulted in the partial opening of the pre-existent fissures and joints, thus imparting to the soil mass a much higher secondary permeability than that measured in the laboratory.
2. The Intact soil along the exposed face of the slope has been significantly broken down by physical

weathering processes within a few years. This breakdown of the soil allows ingress of rainfall and runoff, thus hastening the equalization process. Even if the soil mass has not completely equalized, substantial water pressures may be built up in the discontinuities due to the infiltration.

Due to the lack of complete piezometric data, the pore pressures at failure for the 1973 slide are not precisely known. The data that is available for the area (Thomson, 1978), indicates that the water level is variable, both within short distances along the slope and with time. For example, piezometers approximately 10 metres apart on the north side of the Freeway near the 156th Street bridge (piezometers B and C, Figure 2.3) registered water levels of 2.7 metres and 4 metres below the ground surface on November 18, 1974 and then dropped to 4.3 metres and 5.7 metres respectively, by March 18 of 1975. These large and relatively rapid variations are due to the "perched" nature of the piezometric surface, the highly permeable surface zone which allows easy ingress and evaporation of rainfall and runoff and to the effects of urban land use in the immediate vicinity; i.e. the cultivation of gardens, which allows increased infiltration and evaporation from the soil and the irrigation of the gardens and lawns may result in dramatic changes in the groundwater state (Hamilton and Tao, 1977). Therefore, to assess fluctuations in the water table in the stability analysis of the 1973 slide, a series of piezometric levels were investigated, ranging from a depth of about 2.1 metres below the surface at the crest of the slope to the ground surface (Figure 4.1). Correlations of slope instability with the Edmonton precipitation records (Appendix B) have also been drawn, and are presented in a following subsection.

The strength parameters from the laboratory testing program and those used in the stability analysis are summarized in Table IV.1. Both sets of Intact strength parameters were used in the analysis, namely those representing the main bulk of the material, $c'=15$ kPa and $\phi'=20.5^{\circ}$ and those involving failure through the dark gray fissured clay, in which $c'=7.3$ kPa and $\phi'=18^{\circ}$. The latter parameters were obtained from a linear regression analysis performed on Triaxial Tests #2 and #1c and Direct Shear Tests #3, #3b and #3c. The "fully softened" strength parameters, $c'=0$ and $\phi'=20.5$ and the residual parameters of $c'=2$ kPa and $\phi'=15^{\circ}$ were also considered. An effective angle of shearing resistance of 40° with zero cohesion was used for both the Weathered and Toe zones (i.e. zones I and III, Figure 4.1), as the effective overburden pressure in these regions is less than 42 kPa (Figure 3.6). The effects of weathering in the Toe area (i.e. a change from the Intact to the Toe parameters with time) and the variation of ϕ' in the

Weathered and Toe sections were also investigated.

4.2 Results of the Stability Analysis

A summary of the Morgentstern-Price slope stability analysis undertaken on the 1973 slide (Figure 4.1) is listed in Table IV.2. As stated previously, four piezometric levels (A,B,C and D, Figure 4.1) with the strength parameters outlined in Table IV.1 were analyzed. The following points are noteworthy:

1. The peak Intact strength parameters of c' =15 kPa and ø'=20.5 coupled with the Weathered and Toe peak parameters of zero cohesion and ø'=40° yields factors of safety much greater than 1.0 even with the piezometric level at the surface.
2. The peak Intact strength parameters representing failure through the dark gray fissured clay lenses, c'=7.3 kPa and ø'=18°, with the ø' of 40° in the Weathered and Toe regions yields factors of safety ranging from 1.350 to 0.921 for piezometric surface levels A and D, respectively. A piezometric surface between C and D would result in a factor of safety of unity with these parameters.
3. Assuming peak parameters in the Backscarp and Toe zones with the residual c' of 2 kPa and ø' of 15° in the Intact zone results in factors of safety less than unity for all

piezometric levels considered. The application of the residual strength parameters to the total failure surface would result in lower factors of safety. Therefore, if the residual parameters were being mobilized along the failure surface, a water level significantly lower than 'A' would trigger instability.

4. As with the residual parameters, the fully softened strength parameters of c'=0 and Ø'=20.5° applied along the total slip surface give factors of safety less than unity for all piezometric surfaces considered. With a combination of the fully softened strength parameters in the Intact zone and the peak strength parameters of zero cohesion and Ø'=40° in the Weathered and Toe zones, factors of safety of 1.097 and 0.939 are obtained for piezometric surfaces A and B, respectively.

5. The effect of varying the Ø' from 40° to 35° in the Weathered and Toe areas, using the Intact parameters of c'=7.3 kPa and Ø'=18° and water level 'C' has been investigated in run #5. The factor of safety drops from 1.091 to 1.051, which is not a significant change considering the large change of Ø'.

6. As illustrated by run #6, the application of the "Low Strength" Intact and Toe parameters c'=7.3 and Ø'=18° to both the Intact and Toe sections of the failure plane gives a factor of safety of 1.335 for water level 'A'. It is not known, however, whether the low parameters were operative in the Toe material before the cut was constructed or whether

they are a result of the stress relief and physical weathering brought on by the excavation.

7. Run #7 was undertaken to demonstrate the relative effects on the factor of safety of small variations in c' and Ø' in the Intact zone. It may be noted that the stability is far more sensitive to changes in cohesion than to changes in the effective angle of shearing resistance. For example, lowering the c' from 9.6 kPa to 5.7 kPa while raising the Ø' from 16° to 19.5° causes a drop in the factor of safety of about 0.1. The high sensitivity to cohesion is due to the shallow nature of the slide and the high peizometric surfaces considered, both resulting in low effective normal stresses on the slip surface.

It may be concluded from this analysis that the peak Intact strength parameters (c'=15 kPa and Ø'=20.5°) were not mobilized in the Intact Zone (i.e. Zone II) at the time of failure of the slope. Lower strength parameters, such as those associated with the dark gray, fissured clay (c'=7.3, Ø'=18°), the softened values (c'=0, Ø'=20.5°) or the residual parameters (c'=2.0, Ø'=15°) have controlled the stability. The possibility of each of these strength parameters being mobilized at the time of failure is assessed in the following discussion.

P

4.3 Discussion

The 1973 slide was a first-time slide that failed four years after construction, hence, mobilization of the residual strength parameters at failure could only be brought about by either or both of the following processes:

1. Failure occurred along pre-existing planes of weakness which had already been sheared to the residual strength. Morgenstern (1977) discussed processes such as ancient landslide activity, tectonic folding, valley rebound and glacial shove which result in the in-situ pre-shearing of clays. Many cases of this kind may be found in the literature; e.g. Eigenbrod and Morgenstern, 1972, Palladino and Peck, 1972; Tweedie, 1976.
2. The excavation of the cut generated non-uniform swelling which resulted in in-situ shearing of the clay. This shearing led to a progressive failure mechanism, as was discussed in Chapter I.

Extensive investigations of the Whitemud Freeway site in the fall of 1973 and 1974 and subsequently, in 1977 by the author have not uncovered any evidence to support the theory of pre-existing shear zones at the residual strength. Moreover, the topography of the area before construction was essentially level (Chapter II), eliminating the possibility of ancient slope instability. Although the process of progressive failure is analytically conceivable, there appears to be only one well-documented case history in the literature (i.e. Burland et al, 1977) to support it as a dominant mechanism of slope failure, as has already been discussed in Chapter I. Skempton (1970), James (1970, 1971) and Morgenstern (1977) point out that large deformtions are often necessary to reduce the strength along the entire slip plane to the residual value. James (1970) has quantified this concept in terms of the field strain (Figure 1.2). To reduce ϕ' from the peak to close to the residual value (a dimunition of at least 5°) would require a field strain of approximately 0.2; hence, residual conditions would not be approached until the scarp reached a height of about four feet. The lateral strains due to excavation would never be high enough to bring about these gross deformations.

In order for the "non-uniform swelling" mechanism of progressive failure to be viable, a highly swelling, thin seam of material would have to be sandwiched between two competent strata of non-swelling material; the non-homogeneous swelling would result in differential deformation between the strata, thus causing in-place shearing of the materials. The stratigraphic configuration of the Lake Edmonton deposits, as discussed in previous chapters, is not suitable for this type of process to occur. In summary, there is a lack of evidence which would support the 1973 slide failing at the residual strength.

Although signs of softening in the fissures and joints were not detected in either the laboratory samples or in the field, a failure mechanism proposed by Skempton (1970) involving the localized softening of material in the failure zone may still be reasonable. Skempton (op. cit.) postulates the reduction of the peak strength to the fully softened value in terms of the pre- and post- peak formation of Riedel thrust and displacement shears with subsequent infiltration of water along the opened discontinuities. He has related small pre-slide creep movements to the relatively small displacements required to reduce the strength from the peak to the fully softened state, as opposed to the larger movements necessary for the reduction to the residual. The paper concluded with the following remarks:

> "...In such clays there must be a mechanism of progressive failure and/or softening which takes them past the peak, and just before a first-time slide occurs there is a softened shear zone with many minor shears."

The stability of many slides in the London Clay (James, 1970) and also some in Canada (e.g. Rivard and Lu,1977) have been explained in terms of the mobilization of the fully softened strength, but the actual mechanisms and the time scale involved in this failure process are still unclear. James (1970) and Chandler (1974) investigated cutting slopes which failed from about 10 to 100 years after excavation; although pore pressure equalization is also intrinsically involved, very few failures occurred at less than about 40 years after excavation. Complete softening, involving the total loss of effective cohesion, therefore, appears to be operative on a time scale over decades.

Partial softening, however, resulting in the destruction of some of the peak effective cohesion may have occurred (i.e. the effect may be visualized as a downward shift of the peak strength envelope, but not completely to the origin). A slope stability analysis was subsequently undertaken to assess this partial softening effect. The effective angle of shearing resistance was held constant at 20.5° while the effective cohesion was varied from 10 to 2 kPa. All four piezometric levels were considered. The results of this analysis are as follows:

1. The drop in c' from 10 to 2 kPa with ϕ'=20.5° yielded factors of safety varying from 1.595 to 1.192 for piezometric surface A and 1.428 to 1.032 for piezometric surface B.
2. A factor of safety of unity was obtained for c'=4 kPa and ϕ'=20.5° with the piezometric level C.
3. With the piezometric level at the surface (i.e. level D) the factor of safety varied from 1.078 with c'=10 kPa to 0.745 with c'=2 kPa.

Therefore, from the preceding analysis, factors of safety close to unity are obtained using the partially softened parameters of c' ranging from 2 to 6 kPa and ϕ'=20.5° with the piezometric level between B and D. Hence, with due regard to the preceding discussions, the failure of the 1973 Slide may rationally be explained by the mobilization of the strength parameters of the dark gray, fissured clay and/or the partially softened strength parameters in the Intact zone. For the low normal stress range in this problem, it is impossible to sensibly discriminate between the two sets of parameters and therefore both must be rationally considered.

The mechanism of failure is postulated as follows. Upon rapid excavation of the cut in 1969, the pore pressures immediately dropped in response to the unloading and a large area of soil on the slope face was left exposed to physical weathering processes. In the short term, however, the reduced pore pressures and the as yet "Intact" slope face would result in the stability of the slope, with a factor of safety close to or in excess of the 1.3 using piezometric level 'A' (Table IV.2).

As time proceeded, the pore pressures would equilibrate, but of equal importance is the intense physical breakdown of the Intact material along the slope face due to weathering. Within a few seasons, enough degradation of the soil through freeze-thaw and wet-dry processes would have occurred to dramatically increase the permeability characteristics of the slope. The greater availability of moisture would ,in turn, greatly affect the swelling of the slope, reducing the time of equalization. Coupled with increasing the permeability, the weathering will also cause structural breakdown of the clay in the Toe region with a subsequent reduction in cohesion.

As may be noted in the Edmonton rainfall records (Appendix B), 1972 was an extremely wet year with above normal precipitation in almost every month. This would result in a high groundwater level in the spring of 1973. The slope, at this time, would thus be in a critical state with a relatively high groundwater level and a reduced strength (in terms of cohesion) in the Toe area.

The final mechanism which triggered the slide in August was the periods of heavy rainfall throughout June and in early August (Appendix B). The rainfall was usually concentrated in periods of two to four days, so that maximum infiltration, with little runoff and evaporation would take place. It is, therefore, conceivable that hydrostatic pressures built up in the fissures and discontinuities, represented by a piezometric level between "C" and "D" (Figure 4.1) and triggered the instability. Esu (1966) has undertaken a similar approach with respect to the stability of slopes in Italian jointed clays and has concluded:

> "...Water can seep (or accumulate) through the discontinuities and, at least near the ground surface, the fissured clay can be considered as a permeable medium, even though it has a low coefficient of permeability. In the stability analysis of the slopes in these clays, the hydrostatic pressure of the water

> contained in the discontinuities must be taken into account..."

Morgenstern (1977) has also drawn attention to this "increased permeability" effect, both in the light of the work by Esu (1966) and in the case of the brecciation of the surface layers of the Upper Lias clay (Chandler, 1974). Morgenstern stated:

> "...It influences the permeability profile which in turn affects the rate of convergence of the pore pressures to seepage conditions after initial cutting as well as the responsiveness of the pore pressures in the upper few metres to rainfall."

In summary, the slope failure of August, 1973 along the Whitemud Freeway may be best accounted for by a slip plane mobilizing virtually zero cohesion with $\phi'=40°$ in the Weathered and Toe zones and the dark, fissured clay strength parameters ($c'=7.3$ kPa, $\phi'=18°$) and/or the partially softened parameters ($c'=$ 2 to 6 kPa, $\phi'=20.5°$) in the Intact zone. The rapid increase in hydrostatic pressure brought about by concentrated, heavy periods of rainfall just prior to the failure appears to be the final triggering mechanism. The 1973 slide can therefore be considered as a "delayed failure", involving:

1. The equalization of negative excess pore pressures following excavation, causing a reduction in the

stability with time.

2. Physical weathering processes increasing the mass permeability along the slope face, thus allowing increased infiltration of rainfall and surface water and leading to more critical piezometric conditions.

3. Physical weathering processes resulting in the destruction of cohesion in the Toe material under low normal stresses.

Other slope failures on the Whitemud Freeway (Figure 2.3) may also be correlated with intense periods of rainfall. For example, in 1969, Slide #3 occurred immediately after 41 millimetres of rain fell in the period from the 3rd to the 8th of July. This movement caused a slight bulge of the retaining wall which had been erected following Slide #2. In August, 1969, between the 2nd and the 6th, 83.6 millimetres of rainfall was recorded; this brought about the formation of tension cracks at the top of the slope behind Slide #3. Finally, between the 4th and 5th of September, 1969, 68 millimetres of rain fell; this resulted in the formation of a scarp about 1 metre high at the top of the slope and failure of the retaining wall (Soderberg, 1978). During July, 1972, two slides occurred (Slides #4 and #5) following heavy rainfall in June of that year. Finally, in May of 1977, as a result of record precipitation for that period, gross movements on the order of feet took place in Slide #8.

TABLE IV.1

SUMMARY OF PROPERTIES USED IN THE STABILITY ANALYSIS

MATERIAL	WET DENSITY (kn/m³)	SATURATED DENSITY (kn/m³)	LABORATORY STRENGTH RESULTS				STRENGTH PARAMETERS USED IN THE STABILITY ANALYSIS	
			con.' ≤ 42 kPa		con.' ≥ 42 kPa			
			c' (kPa)	ø'	c' (kPa)	ø'	c' (kPa)	ø'
INTACT	17.9	18.1	15	20.5°	15	20.5°	15	20.5°
			7.3	18°	7.3	18°	7.3	18°
							0	20.5° (fully softened)
							2	15° (residual)
WEATHERED	18.2	18.4	0	40°	17	24.5°	0	40°
TOE	18.2	18.4	0	40°	-	-	0	40°
			7.3	18°	-	-	7.3	18°

Note: Con.' represents the effective confining pressure.

TABLE IV.2

SUMMARY OF SLOPE STABILITY ANALYSIS - 1973 SLIDE

RUN*	SOIL PARAMETERS IN THE ANALYSIS						F.S.	REMARKS
	WEATHERED (I)		INTACT (II)		TOE (III)			
	c' (kPa)	Ø'	c' (kPa)	Ø'	c' (kPa)	Ø'		
1 a	0	40	15	20.5	0	40	1.821	PIEZ. LEVEL 'A'
b	0	40	7.3	18	0	40	1.350	
c	0	40	0	20.5	0	40	1.097	
d	0	40	2	15°	0	40	0.956	
e	0	20.5	0	20.5	0	20.5	0.933	
2 a	0	40	15	20.5	0	40	1.651	PIEZ. LEVEL 'B'
b	0	40	7.3	18	0	40	1.188	
c	0	40	0	20.5	0	40	0.939	
d	0	40	2	15	0	40	0.836	
e	0	20.5	0	20.5	0	20.5	0.793	
3 a	0	40	15	20.5	0	40	1.521	PIEZ. LEVEL 'C'
b	0	40	7.3	18	0	40	1.091	
c	0	40	0	20.5	0	40	0.824	
d	0	40	2	15	0	40	0.748	
e	0	20.5	0	20.5	0	20.5	N.C.	
4 a	0	40	15	20.5	0	40	1.271	PIEZ. LEVEL 'D'
b	0	40	7.3	18	0	40	0.921	
5	0	35	7.3	18	0	35	1.051	PIEZ. LEVEL 'C'
6	0	40	7.3	18	7.3	18	1.335	PIEZ. LEVEL 'A'
7 a	0	40	8.1	17	0	40	0.943	PIEZ. LEVEL 'D'
b	0	40	5.7	19.5	0	40	0.887	
c	0	40	9.6	16	0	40	0.992	

Notes: N.C. means not converging.
I, II and III correspond to zones on Figure 4.1.

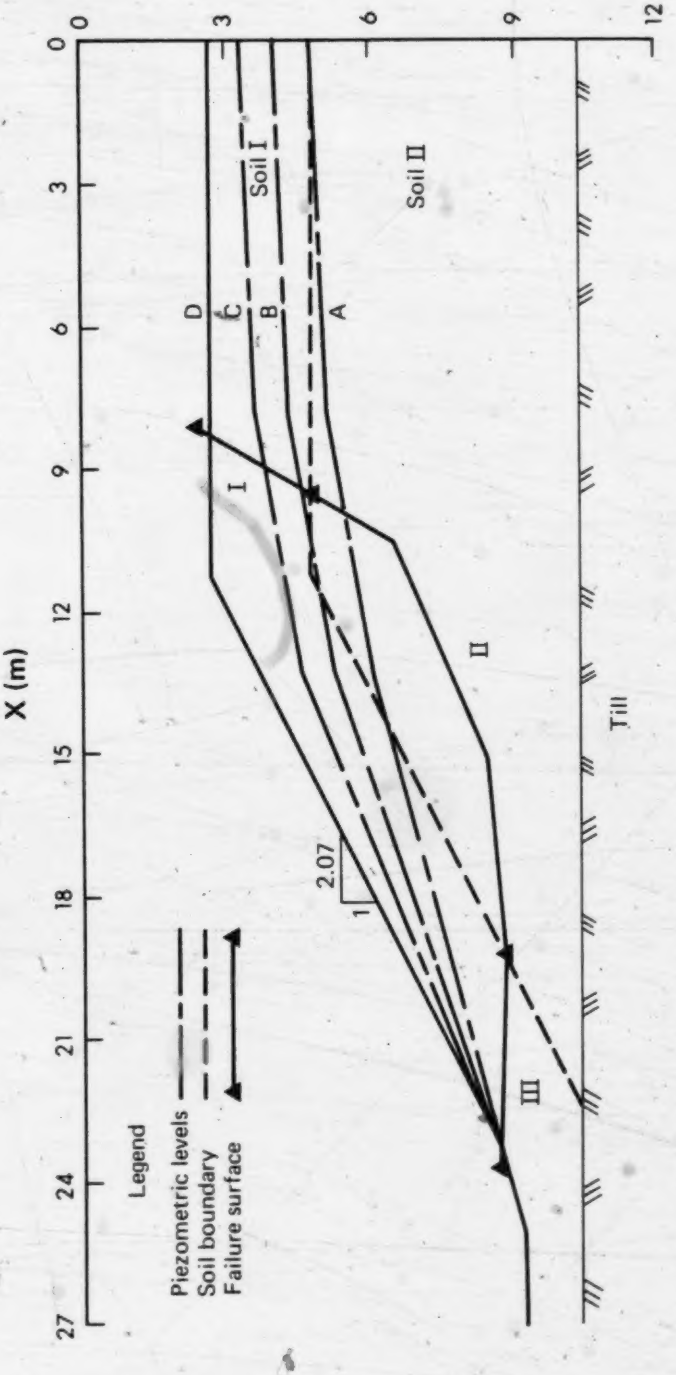

X (m)
0
3
6
9
12
15
18
21
24
27
Y (m)
0
3
6
9
12
Legend
Piezometric levels
Soil boundary
Failure surface
D
C
B
A
Soil I
Soil II
I
II
III
Till
2.07
1

Figure 4.1 **1973 Slide: Non-Circular Slip Analysis Profile**

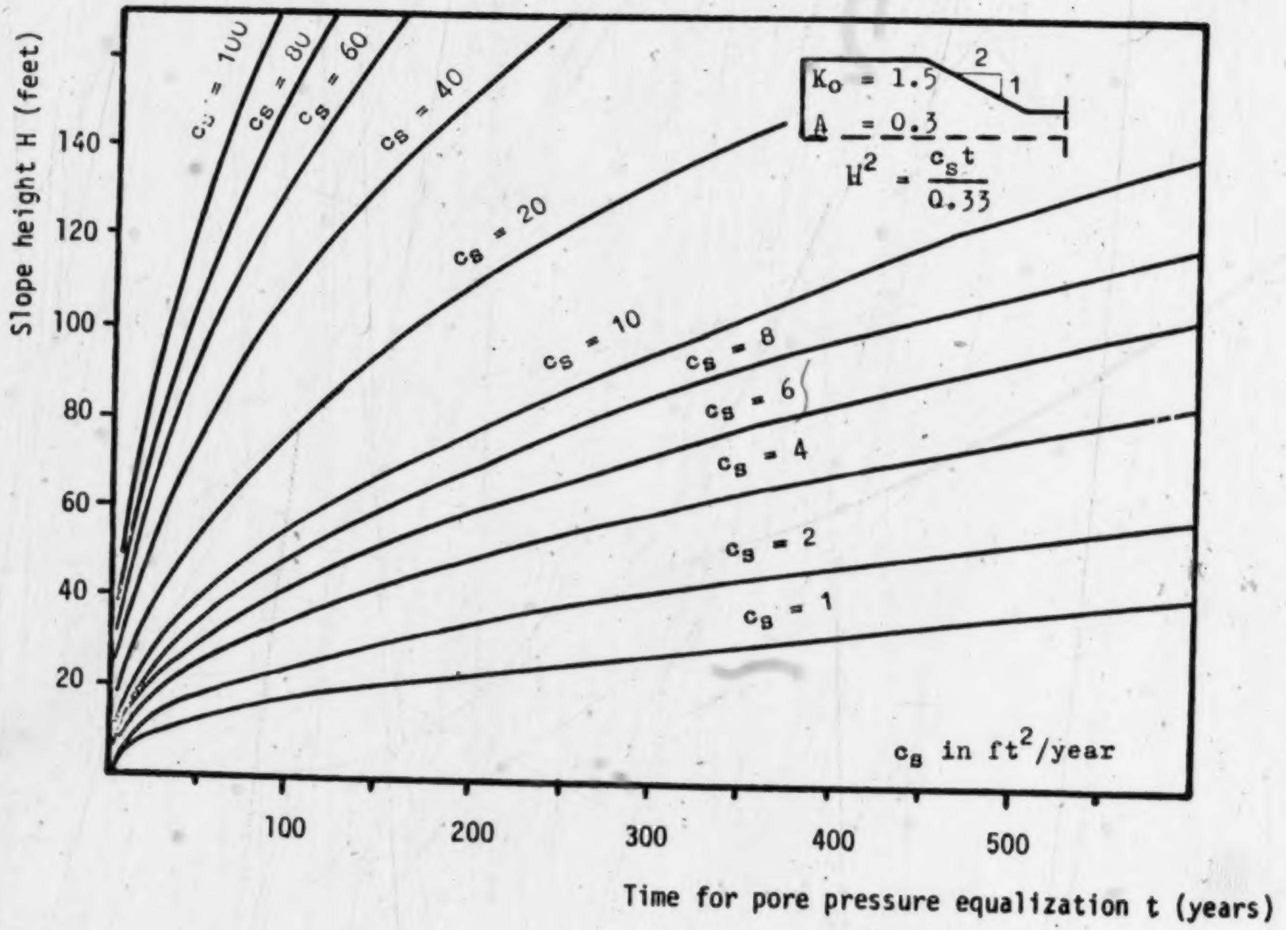


Figure 4.2 Pore Pressure Equalization: Slope Height versus Time for Pore Pressure Equalization for Constant Coefficient of Swelling (After Eigenbrod, 1972).

Chapter V

Conclusions and Recommendations

5.1 Conclusions

The object of this reserach was to investigate the strength and stress-strain/volume change characteristics of the Lake Edmonton deposit, to evaluate the effects of physical weathering and to propose a plausible mechanism of time-dependent failure of the slope. In the light of the results and discussions presented in the previous chapters, the following conclusions may be drawn.

1. The importance of compositional heterogeneity and structural complexity with respect to the strength characteristics of a material cannot be overstressed. The strength of the Intact material was found to depend on the location and orientation of the weaker component in the system, viz., the dark gray fissured clay. The lowest strengths were obtained when failure was confined to these zones.

In a structurally complex deposit, such as the Lake Edmonton sediments, great care must be taken in defining the attitude and occurrence of the various structural elements and assessing their effects on the strength. For example, the characteristic nuggety or blocky structure of the Weathered upper zone had little, if any, effect on the strength above the effective overburden pressure, but below this pressure it controlled the strength; the material behaved as a cohesionless, granular aggregate with shear between the nuggets (Figure 3.6). This is very important in the design of cuts and excavations, as this type of material will have very little strength when unconfined. In fact, caving and sloughing of the unsupported trenches for the counterfort drains was a major problem in the stabilization of the 1976 Slide. The trench walls had to be cut at shallow angles to avoid the short-term failures.

2. The Lake Edmonton material generally displayed marked work-softening behavior upon shear, except when failure was related to structural discontinuities (e.g. Triaxial Test #2 and #9, Appendix A). Such behavior reduces the possibility of progressive failure as discussed in Chapter I. The drained Triaxial Tests undertaken demonstrated that excellent correlations of volume change characteristics with the modes of failure exist; for example, Triaxial Test #9 (Appendix A), which involved shear around the nuggets, dilated much more than Triaxial Test #8, which involved shear through the nuggets. A direct correlation of volume change and mode of failure could not be postulated for the direct shear tests. The contraction/dilation characteristics were found to be more dependent on the normal stress applied.

3. The processes of physical weathering such as repeated cycles of wet/dry and freeze/thaw appear to increase the

strength of the intact lumps or nuggets through a net consolidation or densification effect, but to decrease the mass strength of the material, reducing it essentially to a cohesionless, granular aggregate. Before further research has been undertaken, however, it would not be prudent to apply this effect in a general manner to other overconsolidated Pleistocene deposits.

4. Effects of the softening process as described by Terzaghi (1936) have not been apparent in this material. Detailed observations of the fissures and joints in the laboratory and the field have revealed surfaces which were moist, but essentially stiff and intact. It appears that the complete softening of the material is operative on a much longer time scale. Partial softening, possibly due to a displacement induced mechanism (e.g. Skempton, 1970), is likely, however, to have occurred.

5. The nature and severity of the climate have a great influence on the effects of weathering. For example, in the United Kingdom the depth of frost penetration is limited and moisture is available all year round; therefore, physical weathering processes would proceed in a generally damp enviroment and at a relatively slow rate. James (1970) in fact states:

> "...It is believed that over the length of time involved in this present study of cutting failures, say 120 years, geological weathering on soils would be of relatively minor importance."

In Western Canada, however, particularly in the Edmonton region, the climate is much more severe, with long cold winters and warm summers. Frost penetration to depths of 2 to 2.5 metres can result in severe damage to a soil fabric in a matter of a few seasons. The desiccation of the soil also results in structural breakdown and the formation of a "desiccation crust" at the surface. Therefore, it is proposed that in a harsh climate, such as in Western Canada, the effects of physical weathering on a freshly exposed cut or excavation cannot be ignored on an engineering time scale.

6. The results of the stability analysis undertaken on the 1973 Slide have strengthened a number of the conclusions postulated previously, viz:

    a) The stability of the cut appears to be dependent on the strength associated with the lenses of dark gray fissured clay as well as partial softening in the Intact zone. Unacceptably high factors of safety result for all possible groundwater conditions if the higher peak strength parameters ($c' = 15$ kPa, $\phi' = 20.5°$) for the Intact material are used.

    b) The importance of physical weathering on an engineering time scale has been proven. Physical breakdown on the slope face has greatly increased the mass permeability allowing easy infiltration of surface runoff and precipitation. The results of this have been a reduced pore pressure equalization time and the realization of more critical groundwater levels. The weathering has also reduced the effective cohesion of the Toe material in the low normal stress range; in a shallow slide with a high piezometric surface, the stability is very dependent on this effective cohesion.

7. The stability analysis has emphasized that laboratory testing programs should be conducted in the stress ranges encountered in the field problems. For example, it would be grossly incorrect to apply the parameters of $c'=17$ kPa and $\phi'=24.5°$ (Table IV.1) to the Weathered zone in the slip analysis, as failure is actually occurring below the stress range for which these parameters are applicable.

8. The correlations of landslide activity with peak periods of rainfall have shown that rapid fluctuations in the groundwater level can have a profound influence on stability and act as a final triggering mechanism. The need for an accurate assessment of possible groundwater level fluctuations is thus clearly evident.

9. Finally, it should be emphasized that when dealing with a heterogeneous, complex deposit such as the Lake Edmonton sediments, one should concentrate the investigations on weaknesses, such as incompetent components or beds and structural discontinuities rather than on the more competent materials.

5.2 Recommendations

1. This thesis demonstrates the importance of having accurate, complete piezometric data for a slope or excavation. Piezometers should be monitored regularly from the time of construction to see if the pore pressures are responding in accordance with the design calculations.

2. The nature, occurrence and orientation of fissures are very important in many geotechnical problems, but very little is actually known about their origin and the factors that control their distribution in soil masses. Research into this aspect would prove to be extremely valuable.

3. It is recommended that a detailed investigation be conducted on the impending slope failure on the north side of the Whitemud Freeway near the 156th Street Bridge (Figure 2.3), possibly in conjunction with a stabilization scheme. The investigation would involve:

   1. In-situ direct shear tests conducted on various levels in the slope would supplement the shear strength data already collected and analyzed. Marsland and Butler (1967) obtained valuable results from such tests conducted on the stiff fissured Barton Clay.

   2. A detailed examination of the incipient failure zone may conclusively indicate which processes or mechanisms are operative in a slide before catastrophic failure; i.e. is the failure zone composed of a softened zone of Riedel thrust shears

not as yet having coalesced (Skempton, 1970) or is the failure confined to one thin, continuous shear? Samples taken directly from the shear zone and tested in the laboratory would substantiate the field observations.

Such an investigation, therefore, would be an invaluable contribution in dealing with the mechanisms involved in time-dependent failures in stiff fissured clays.

4. In-situ permeability tests conducted on the Whitemud Freeway cuts would quantify the increase in mass permeability brought about by weathering processes . Chandler (1974) has undertaken such tests in the Lias Clay and has obtained excellent results.

5. Creep or relaxation tests should be conducted to evaluate the time-dependednt deformation properties of the Lake Edmonton sediments and to check if the short-term laboratory peak strength may actually be by-passed. A recent publication by Nelson and Thompson (1977) has indicated that this is possible in overconsolidated clay, and tests by Fontoura on coal and Tse on granite and limestone have shown that the by-pass effect, in fact, occurs in rock.

6. Finally, tests to quantitatively assess the effects of physical weathering on a soil mass in a cold, continental climate should be undertaken. Cook (1963) investigated the effects of closed system freeze-thaw cycles on a compacted, highly plastic clay. He found that the majority of the

strength loss was due to a reduction in cohesion, with the angle of internal friction not being greatly affected; a strength reduction of almost 50% was noted after just three cycles of freeze-thaw. Carefully controlled freeze-thaw tests on the heterogeneous Lake Edmonton deposits would supplement the work of Cook; temperature gradients commonly occurring in the field could be applied one dimensionally to the sample, thus simulating actual field conditions. The structures so-formed could be observed and pertinent soil properties such as densities and moisture contents could be computed after completion of the tests.

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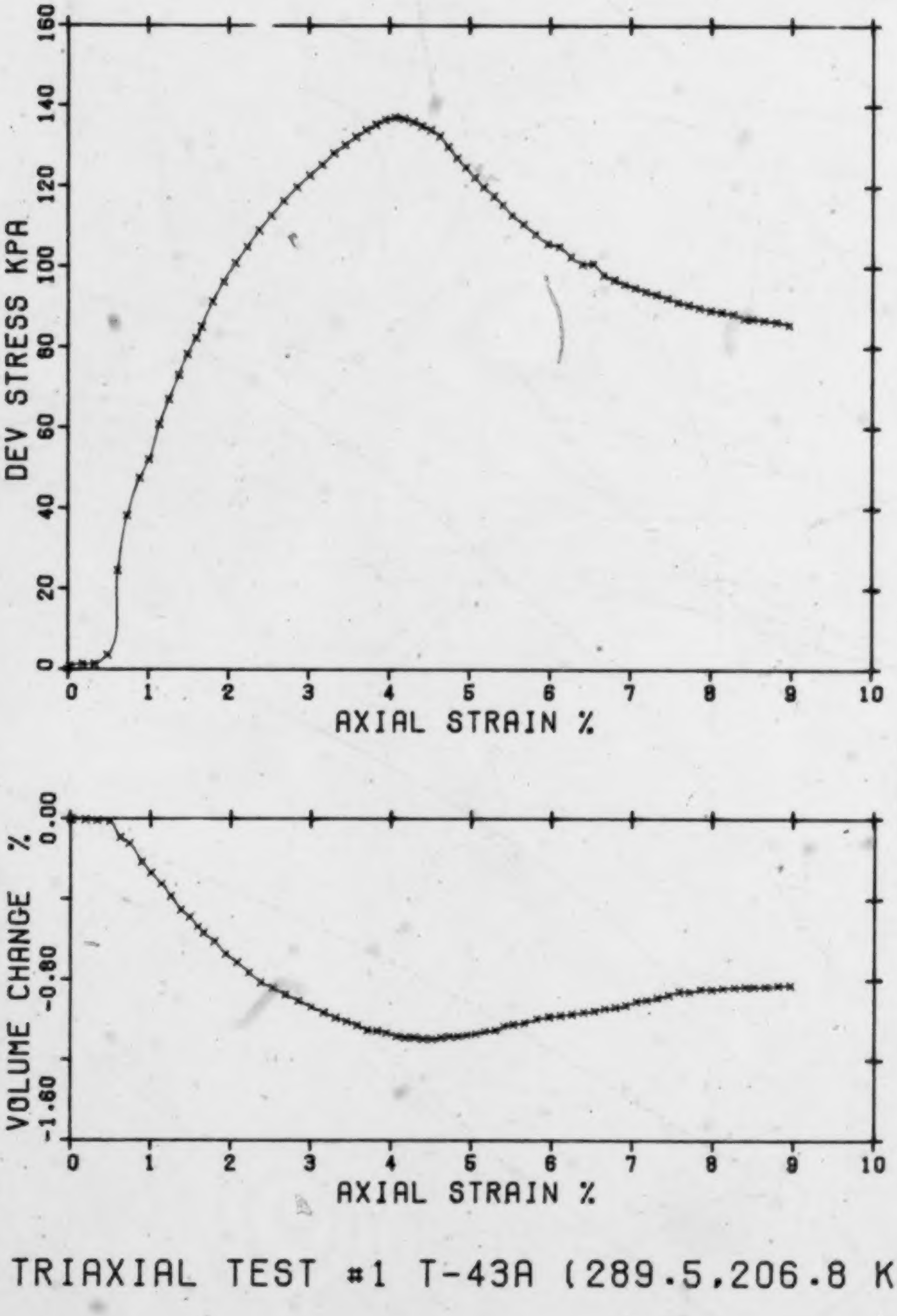
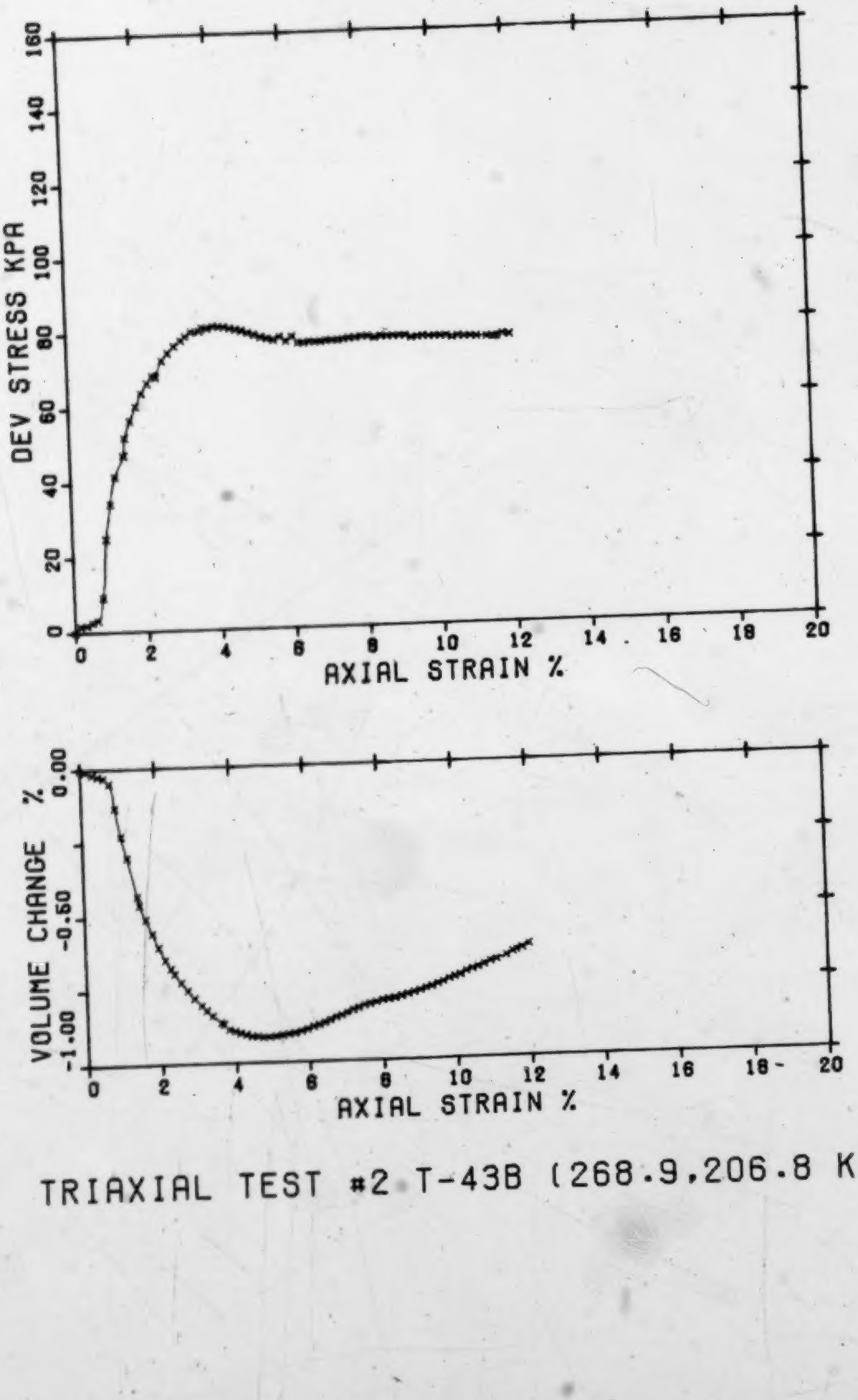
APPENDIX A

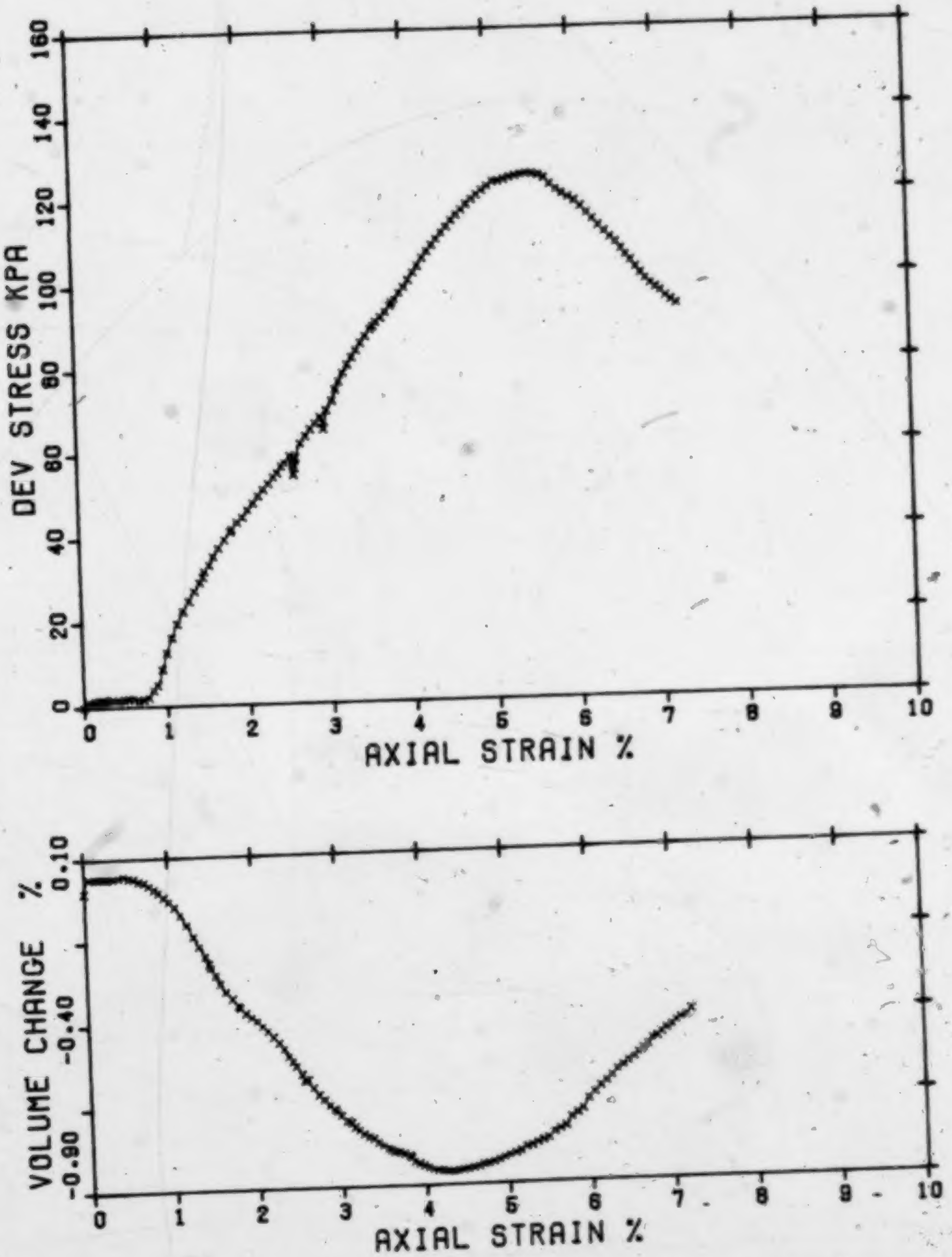
Plots of Test Results



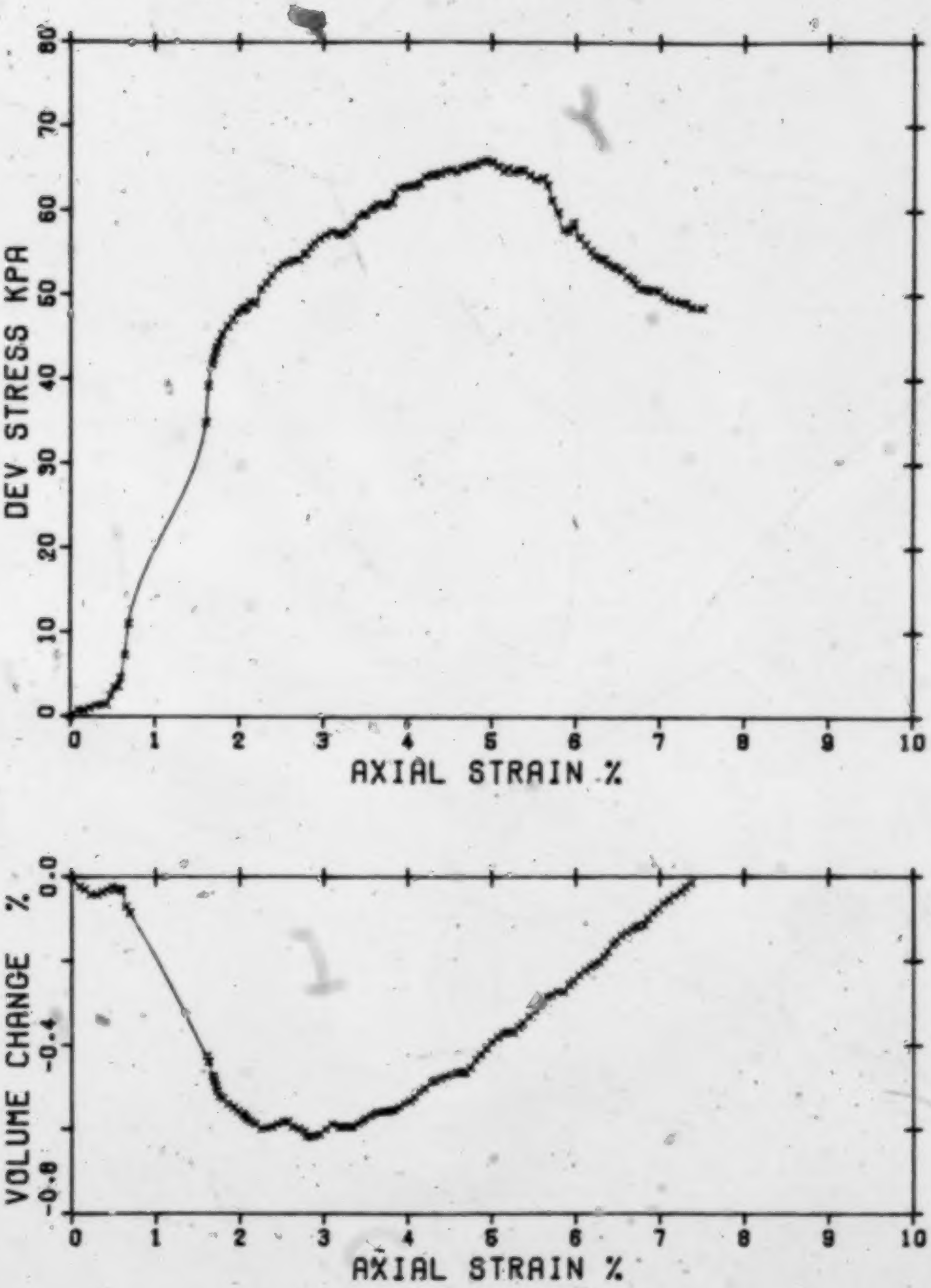
TRIAXIAL TEST #1 T-43A (289.5,206.8 KPA)



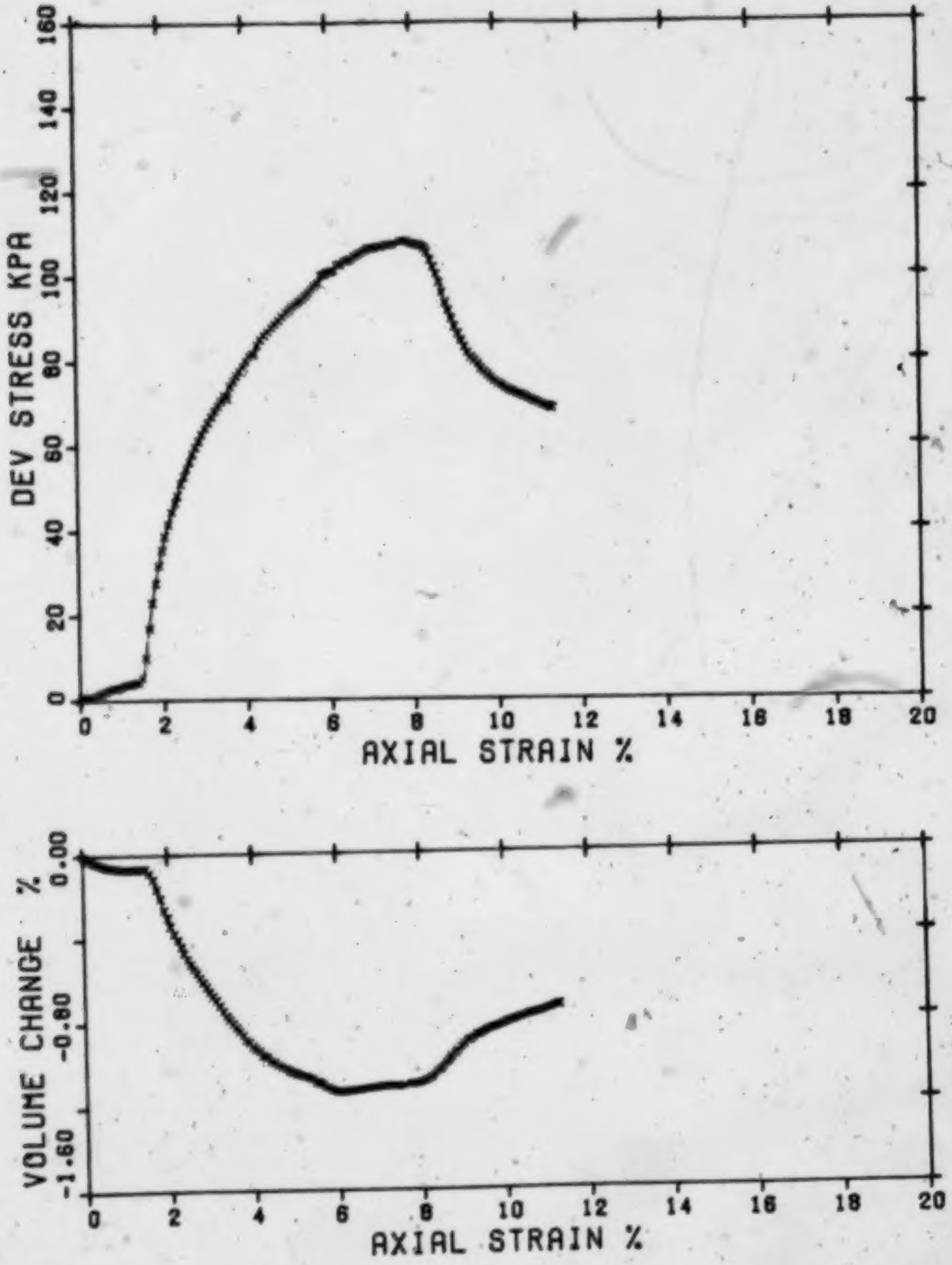
TRIAXIAL TEST #2 T-43B (268.9,206.8 KPA)



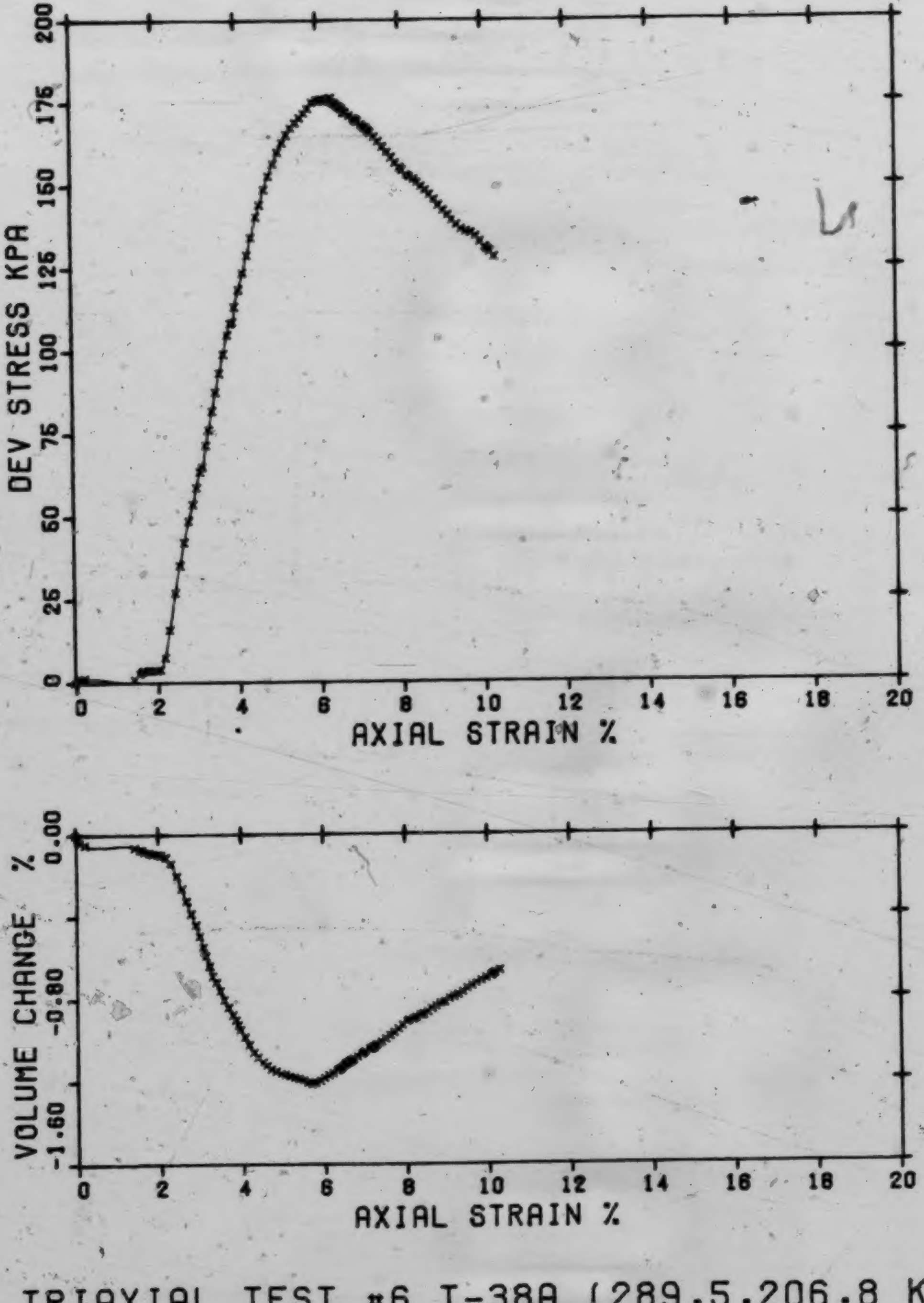
TRIAXIAL TEST #3 T-43C (248.2,206.8 KPA)



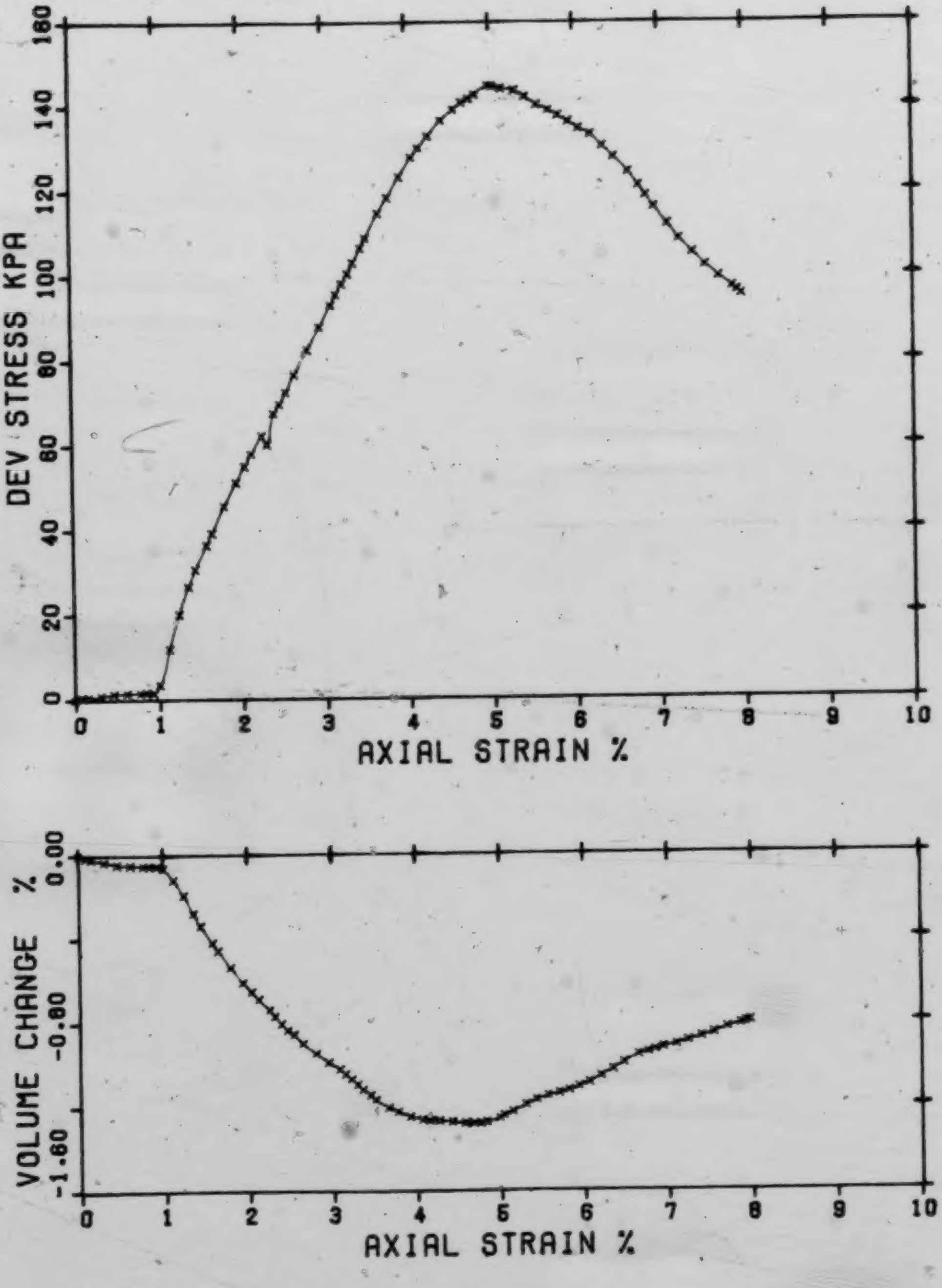
TRIAXIAL TEST #4 T-44A (227.5,206.8 KPA)



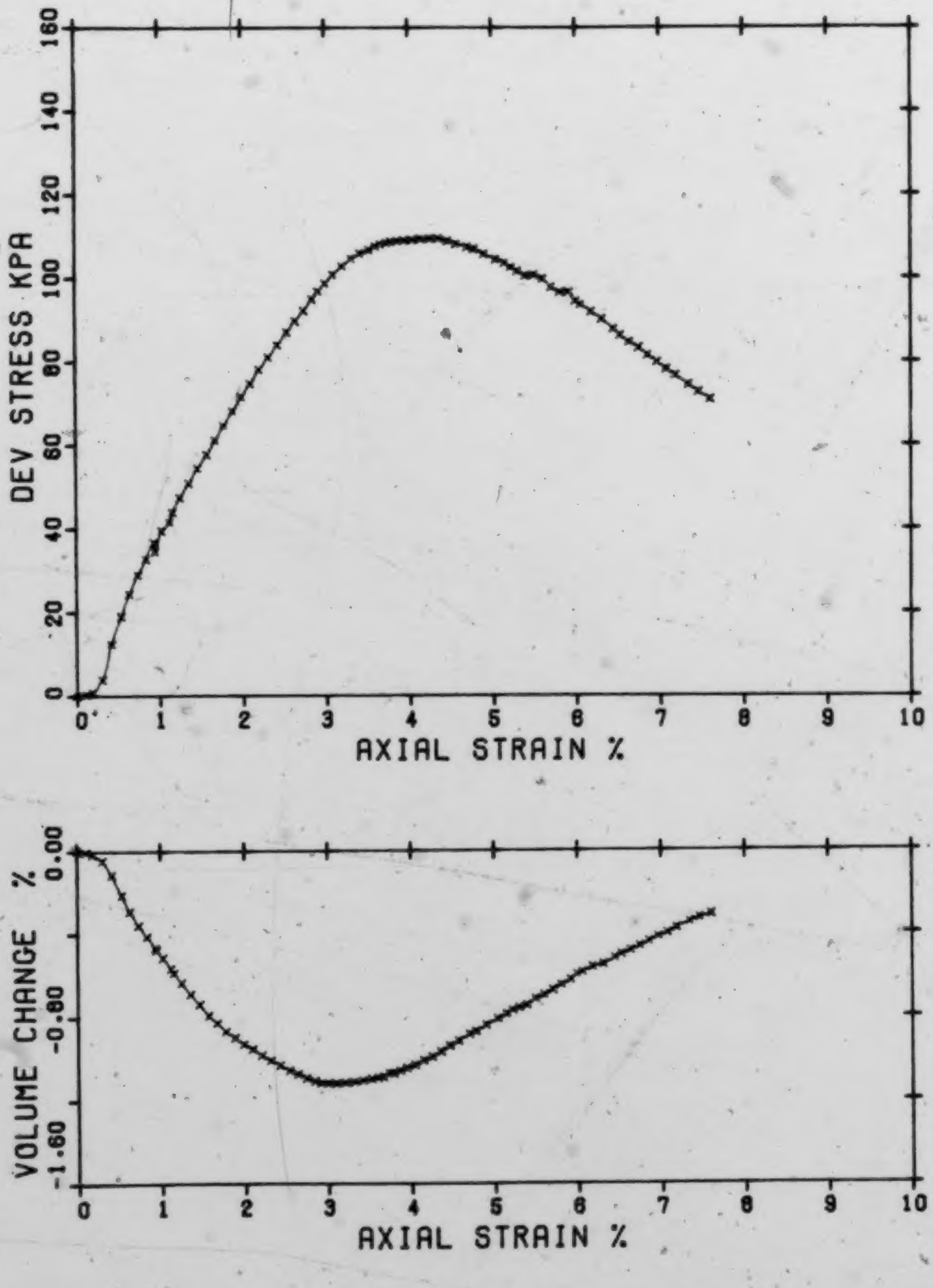
TRIAXIAL TEST #5 T-42A (258.5,206.8 KPA)



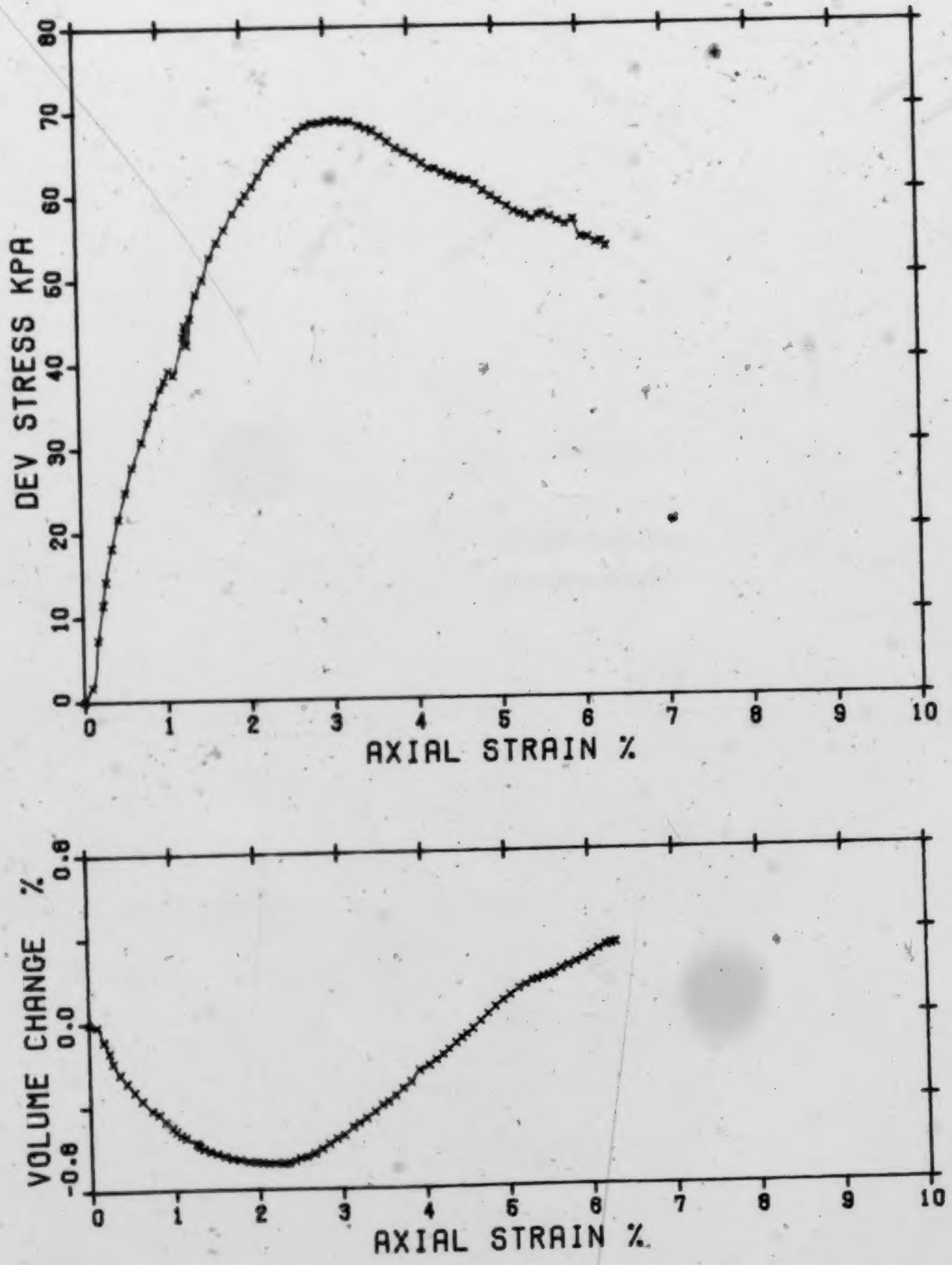
TRIAXIAL TEST #6 T-38A (289.5,206.8 KPA)



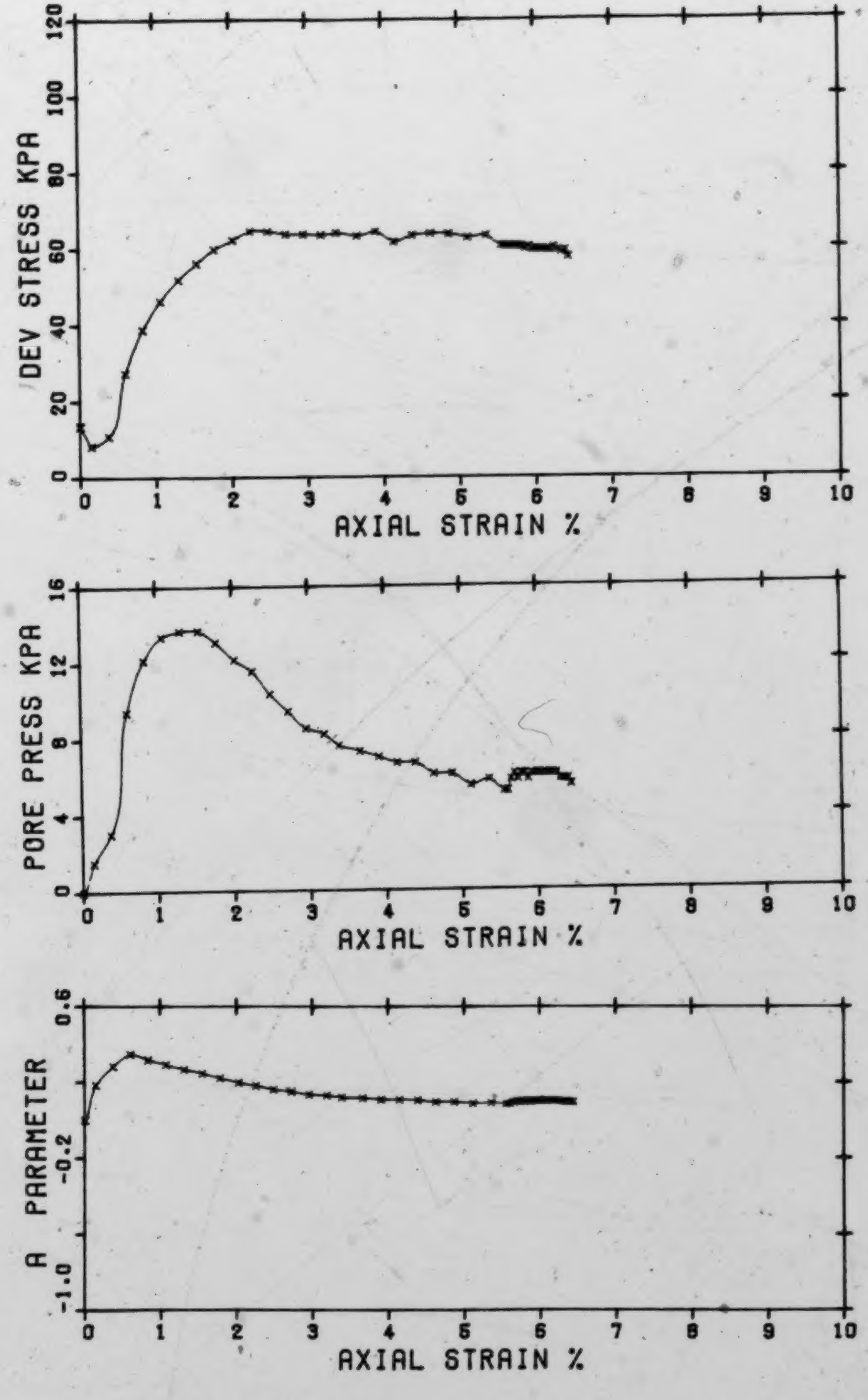
TRIAXIAL TEST #7 T-40A (268.9,206.8 KPA)



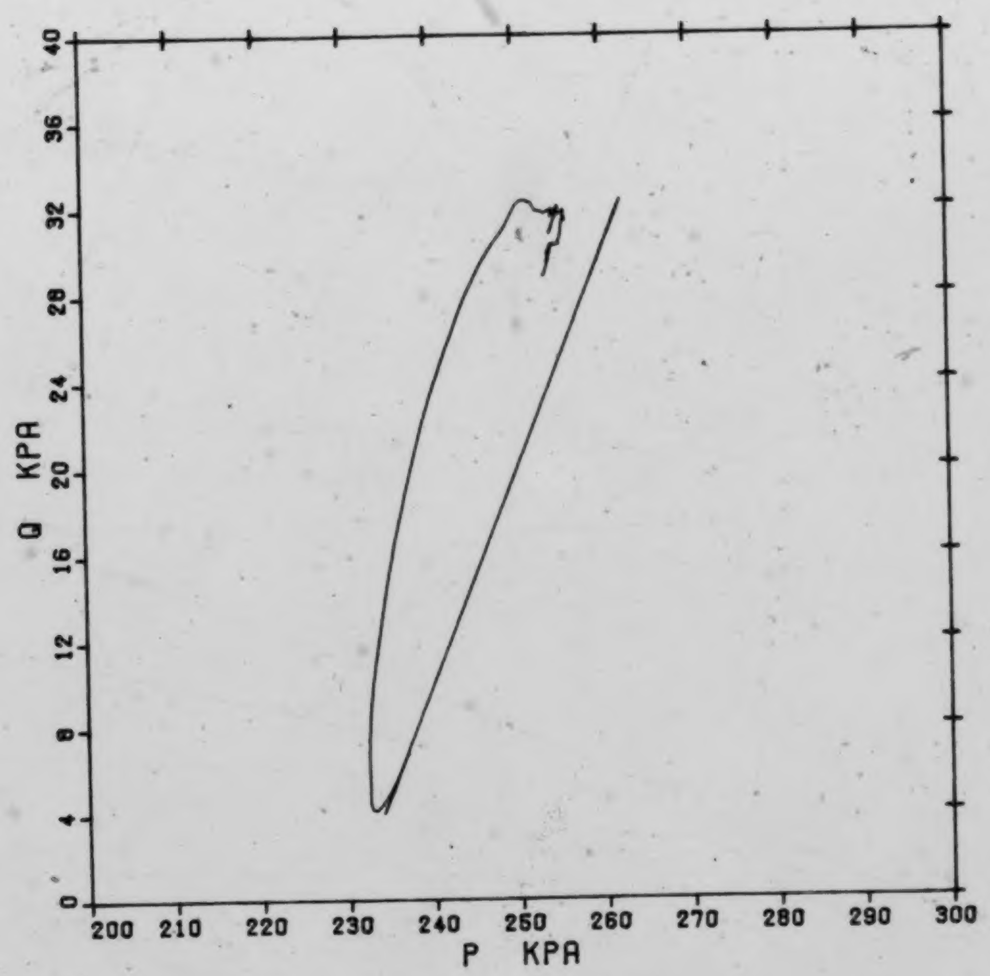
TRIAXIAL TEST #8 T-38B (248.2,206.8 KPA)



TRIAXIAL TEST #9 T-39A (227.5,206.8 KPA)



TRIAXIAL TEST #1D T-41D (230.0,200.0 KPA)



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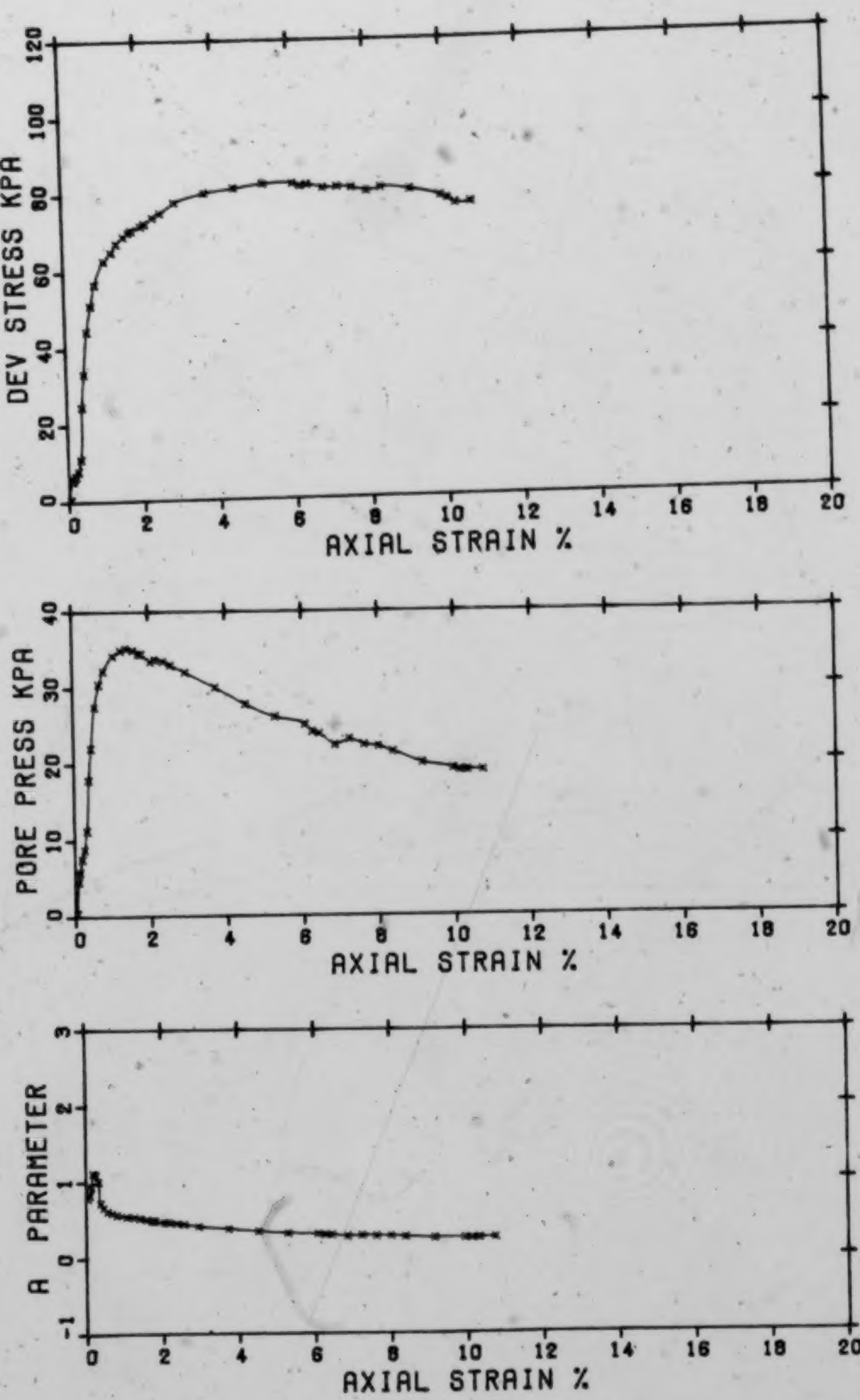
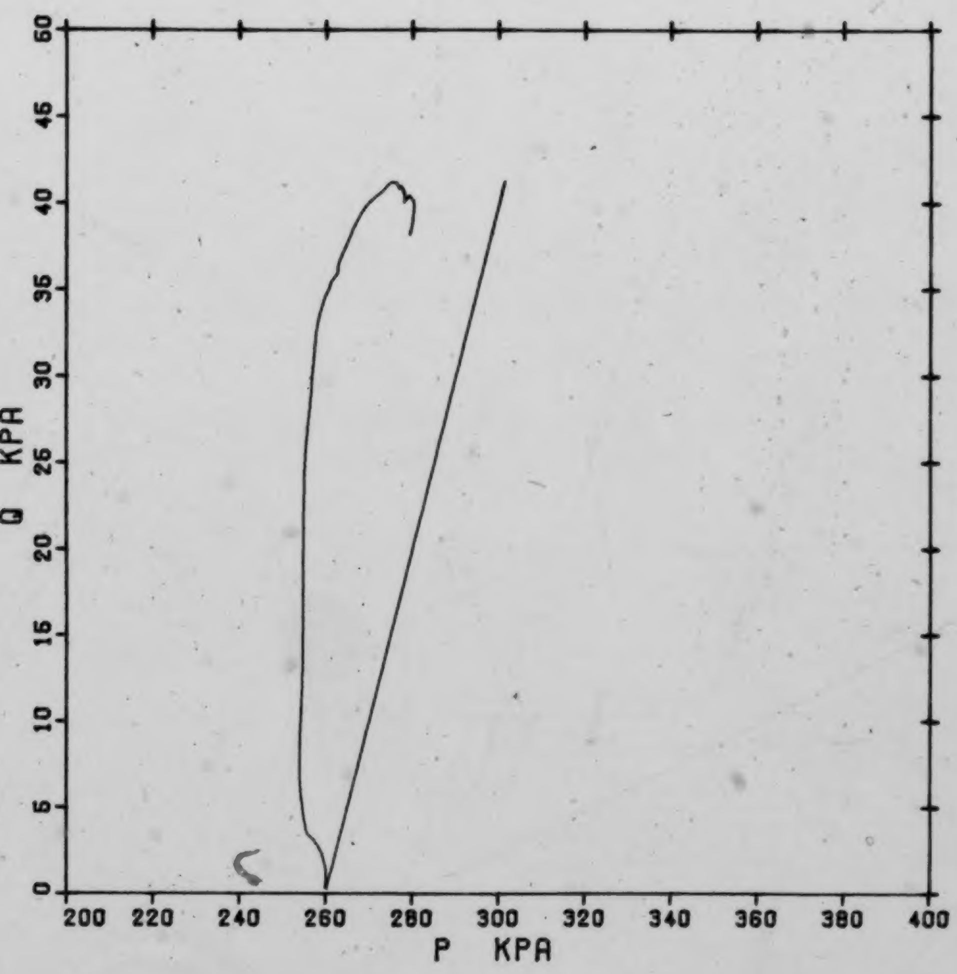


TRIAXIAL TEST #2A T-30A (260.0,200.0 KPA)



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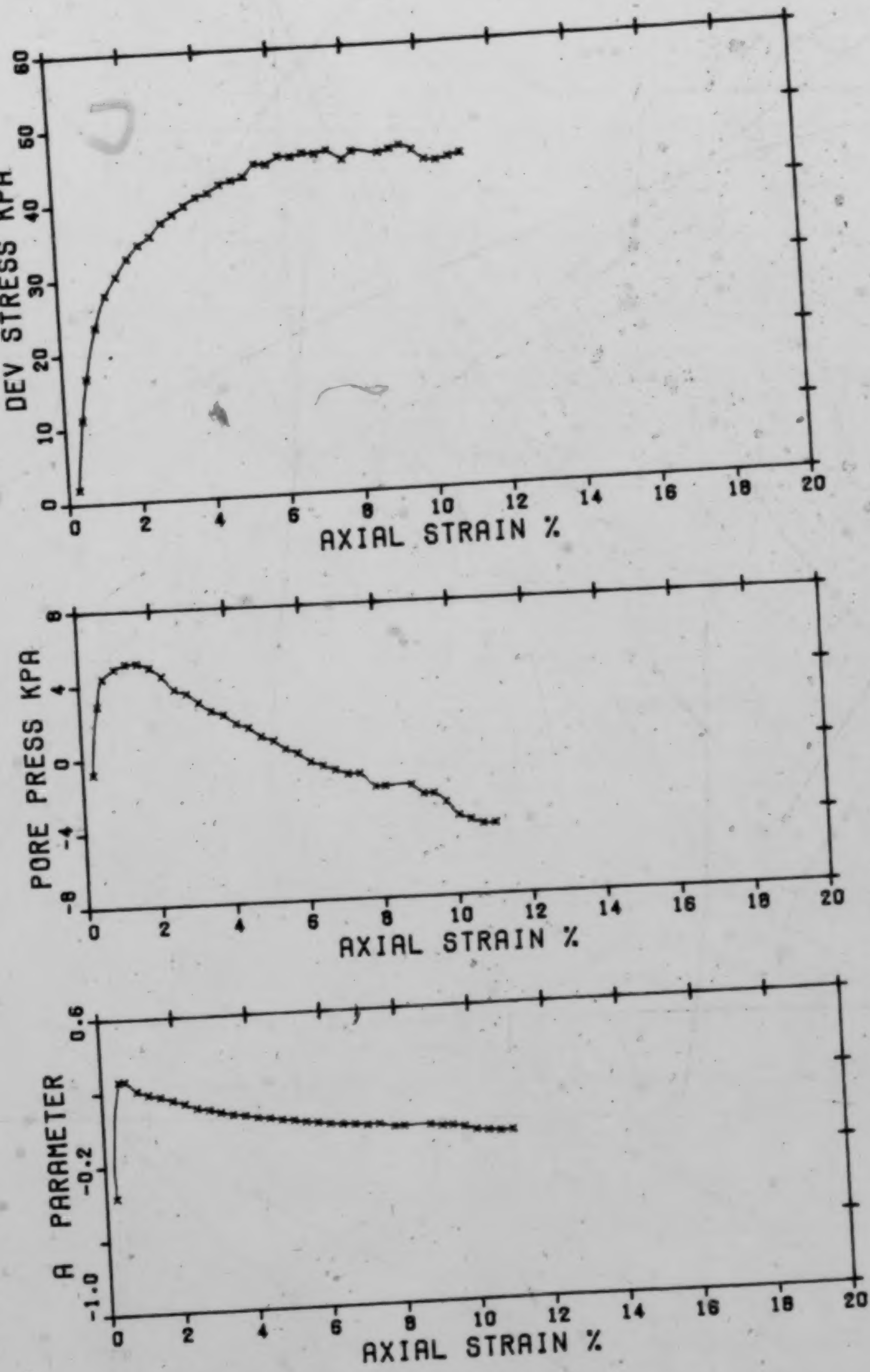
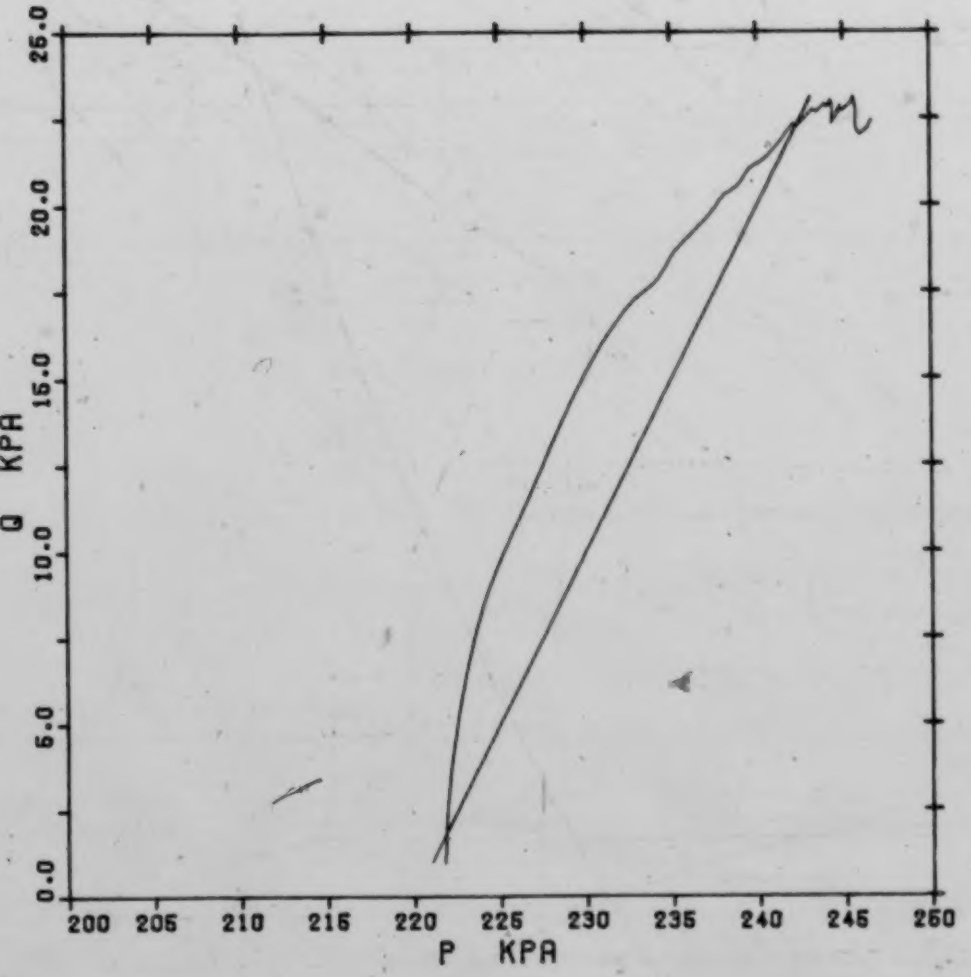


TRIAXIAL TEST #2C T-30C (220.0,200.0 KPA)



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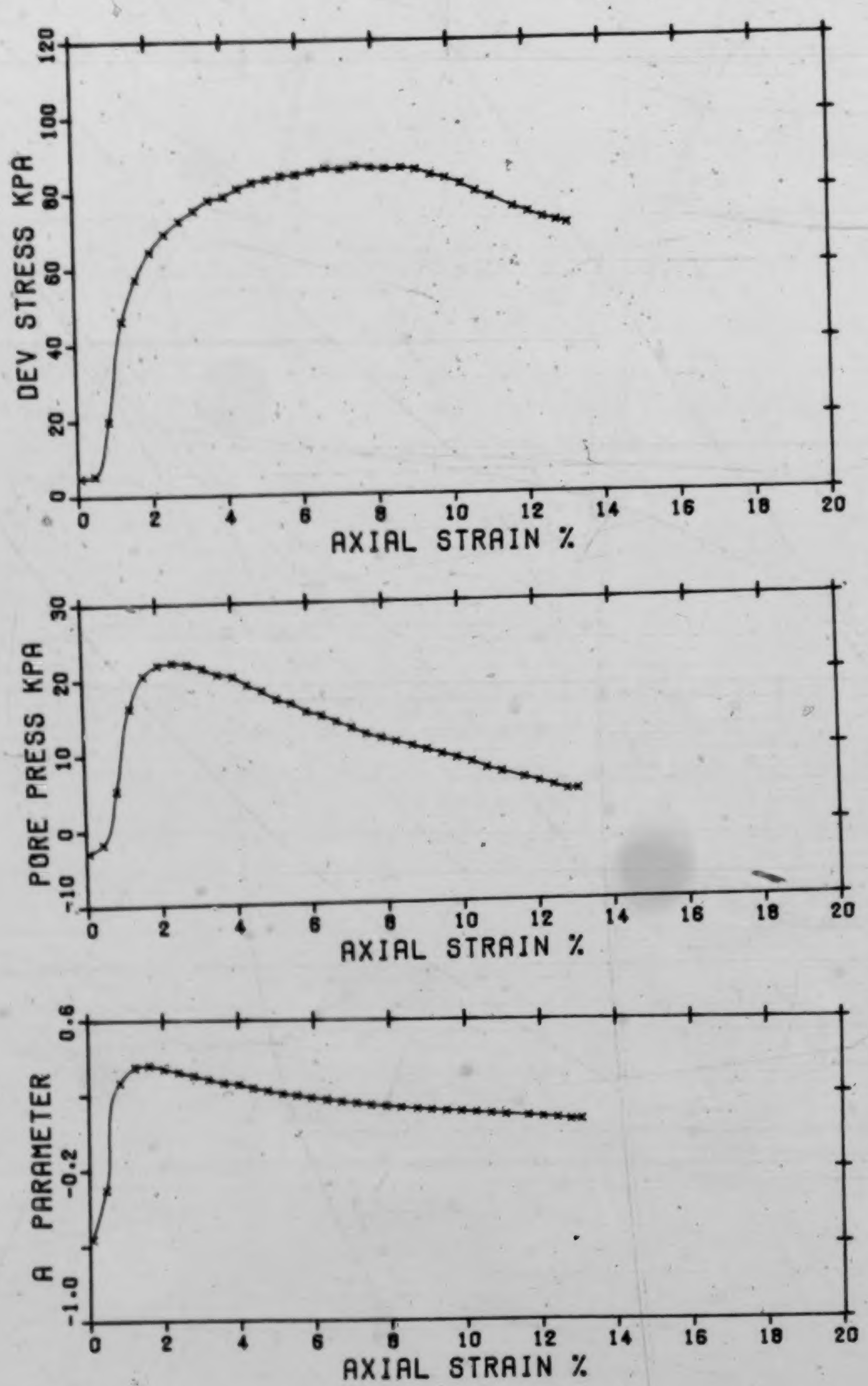
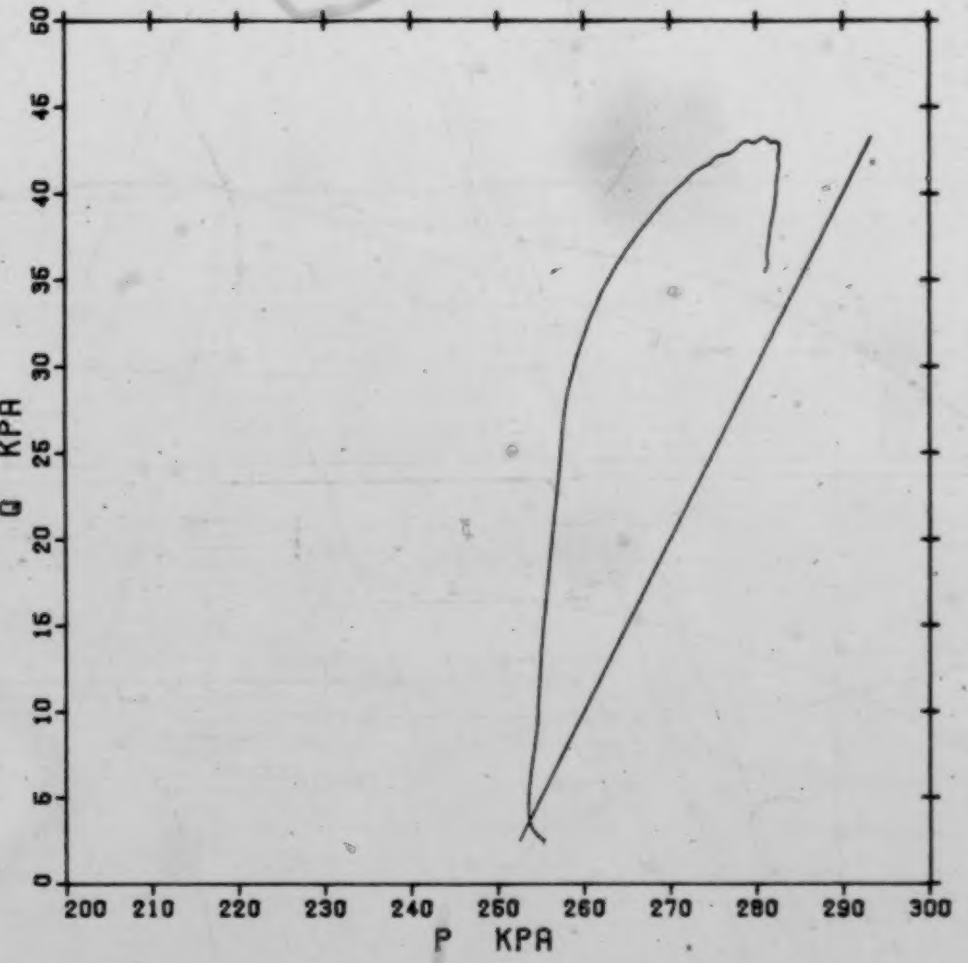
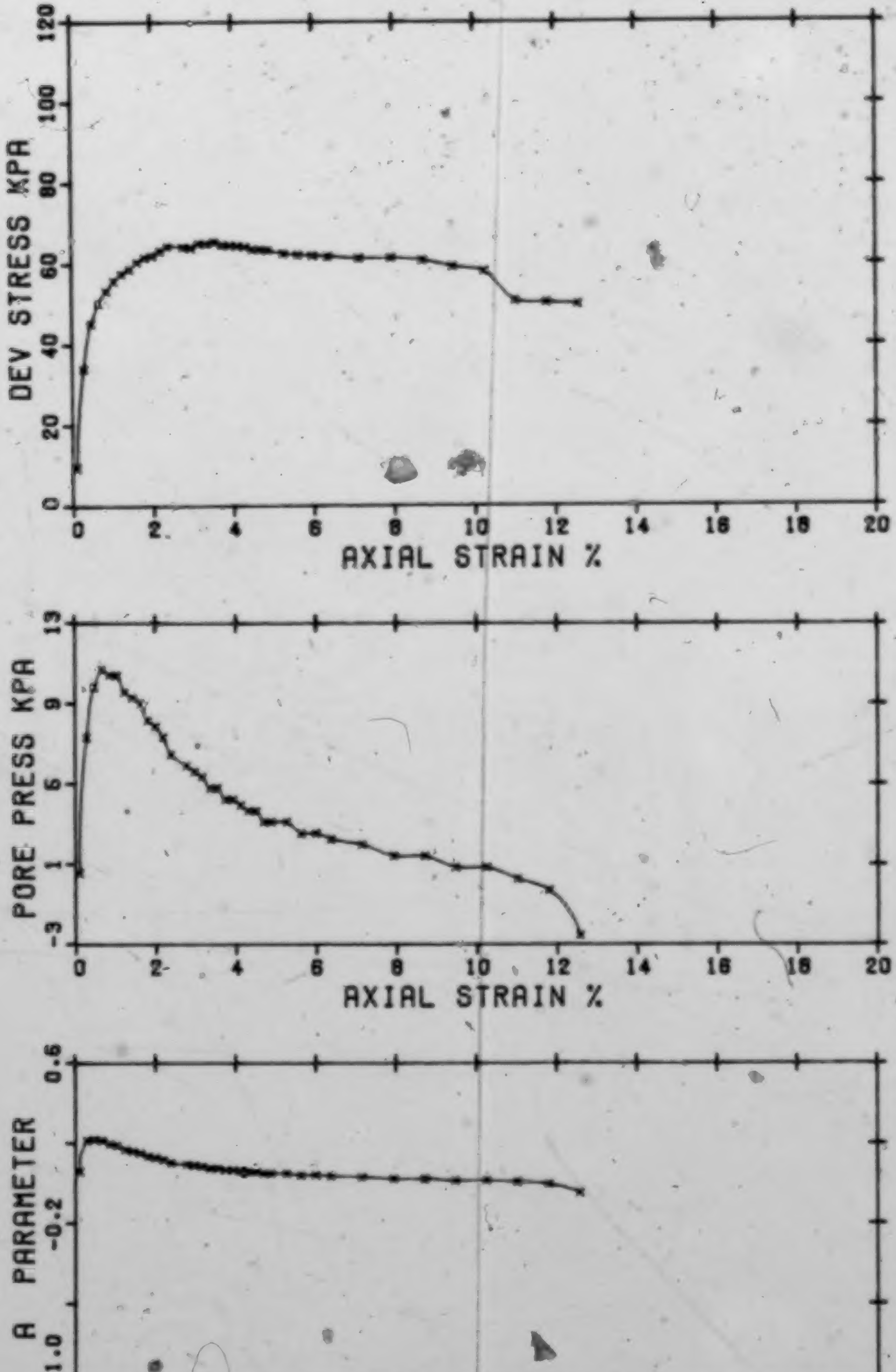


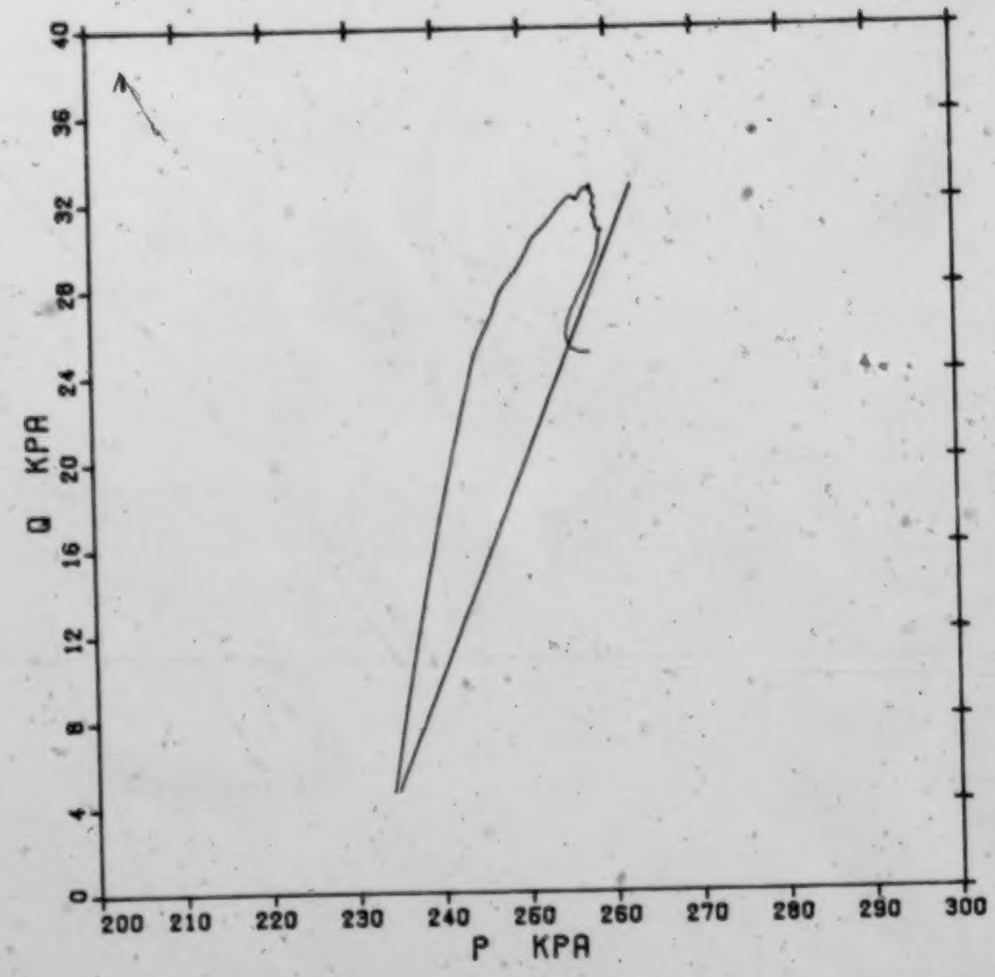
TRIAXIAL TEST #3B T-29C (250.0,200.0 KPA)



TRIAXIAL TEST #3B T-29C (250.0,200.0 KPA)



TRIAXIAL TEST #3C T-29B (230.0,200.0 KPA)



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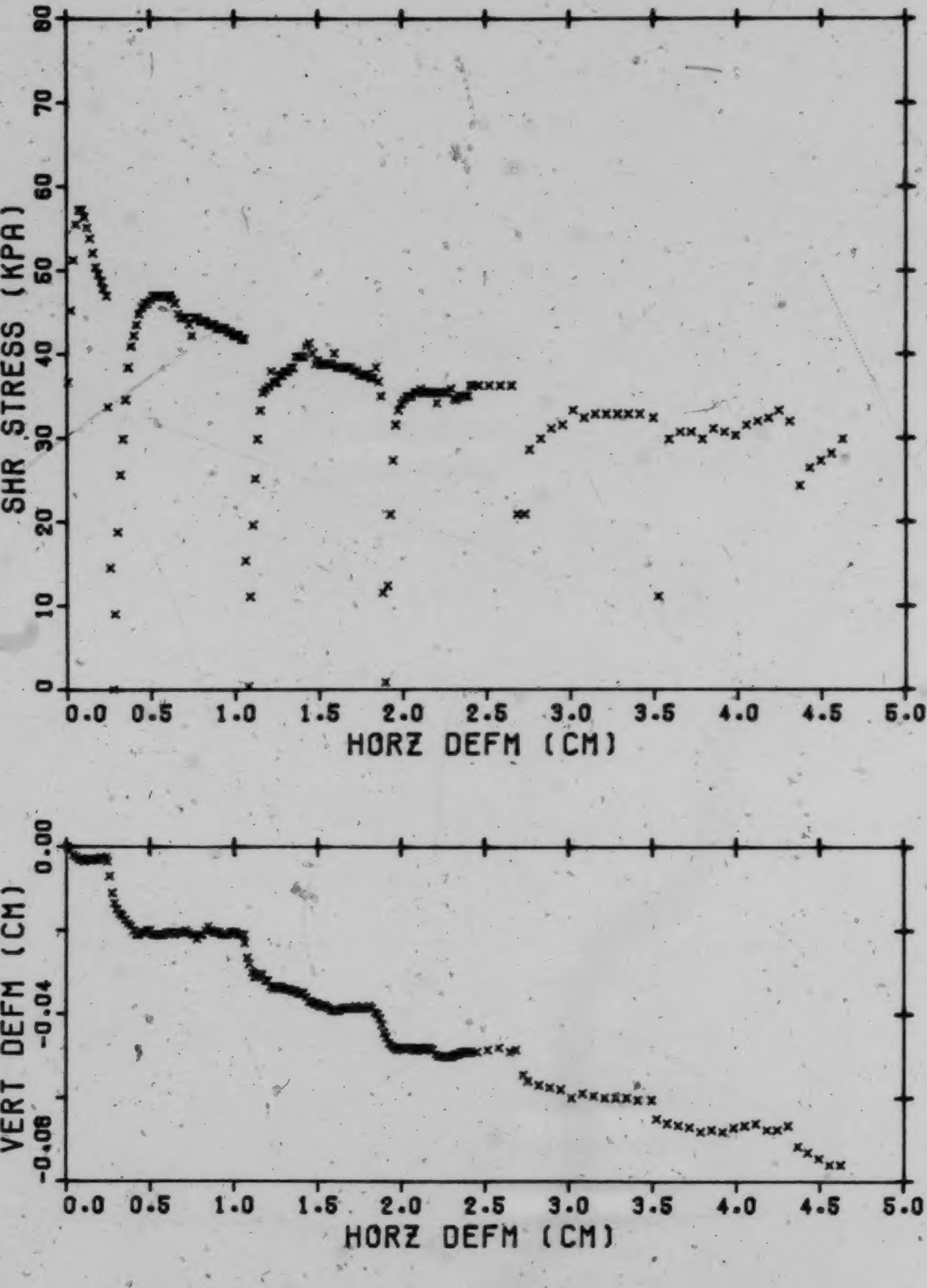
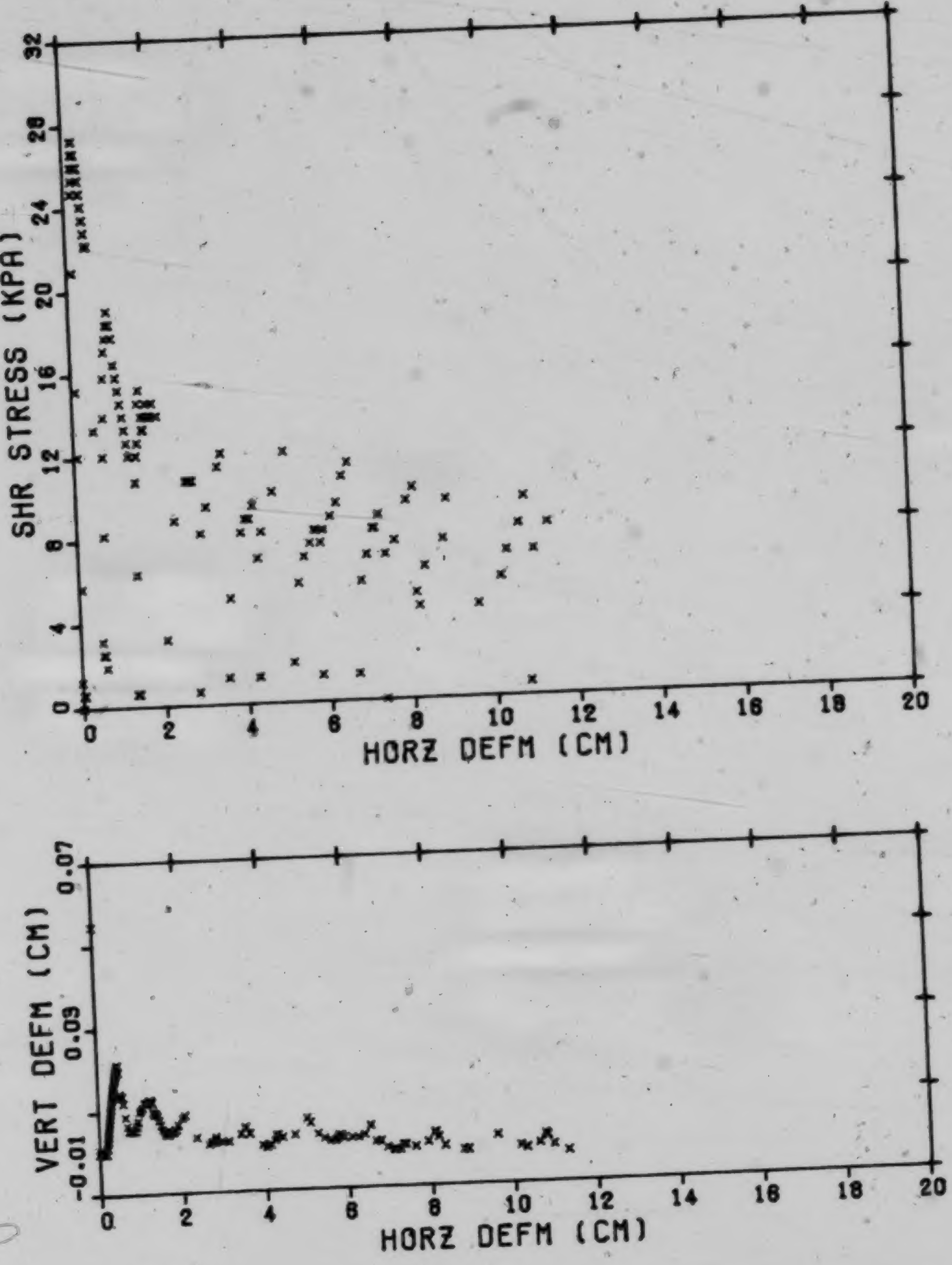


DIRECT SHEAR TEST # 1 SAMPLE T-43(A)



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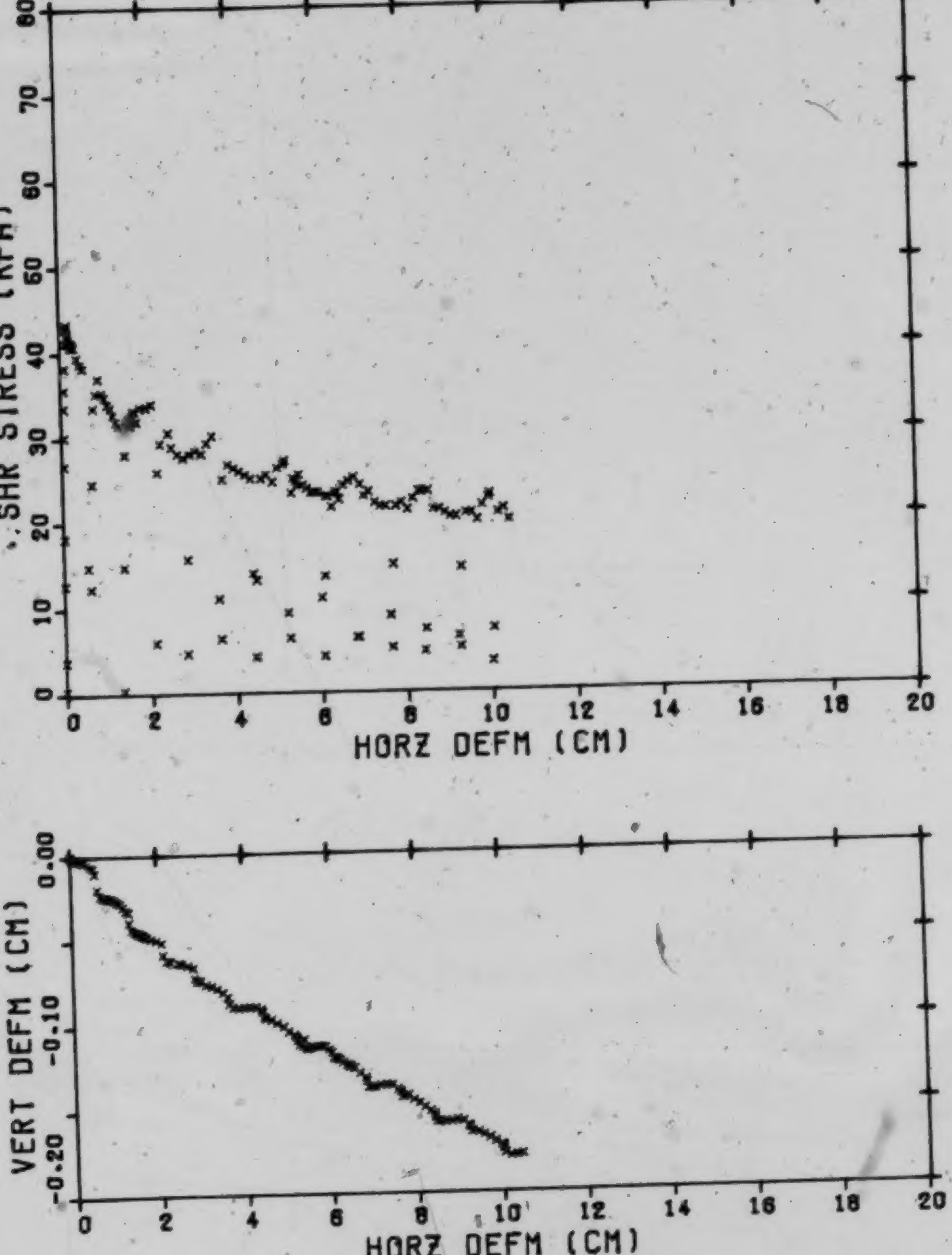

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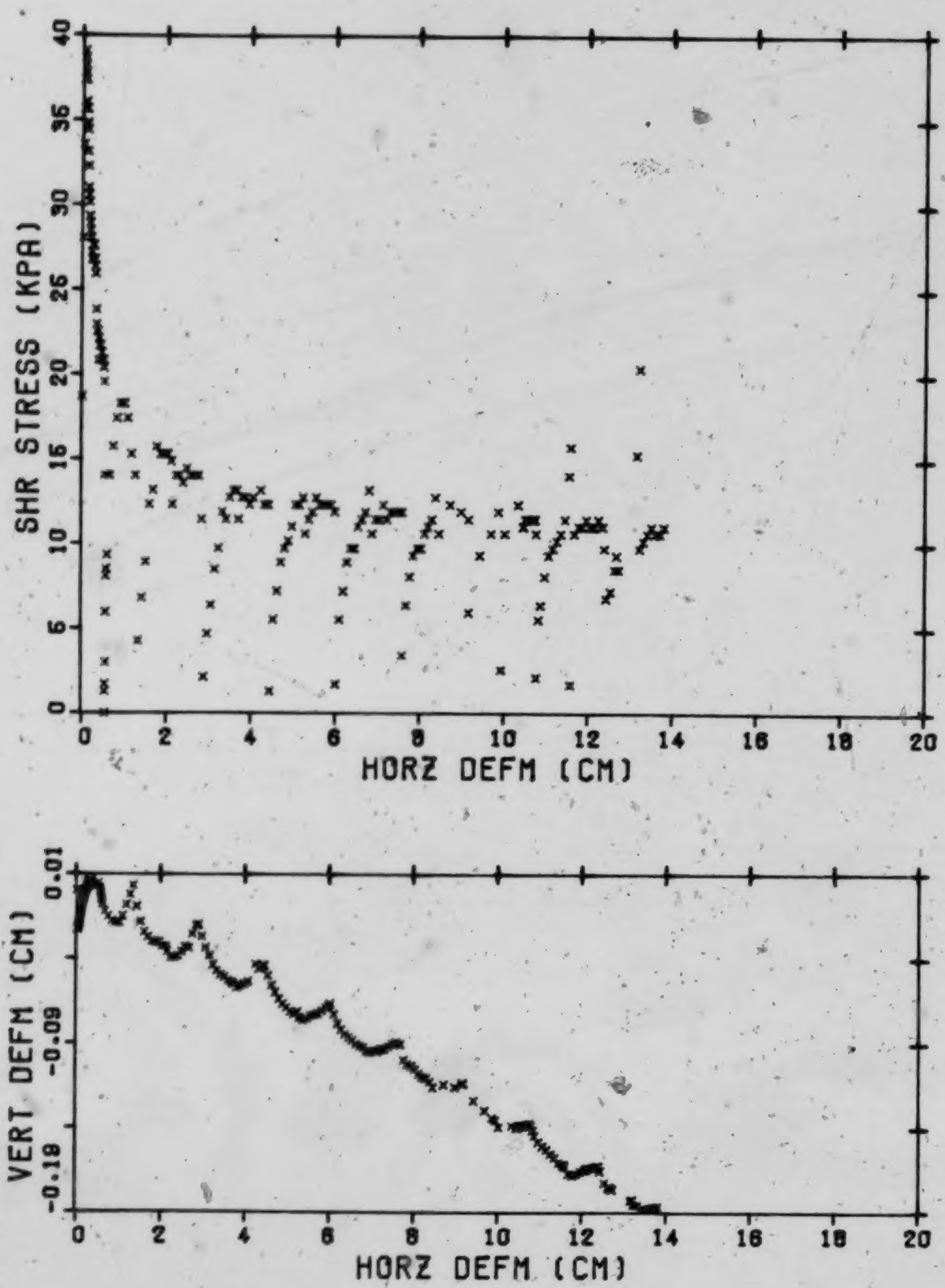

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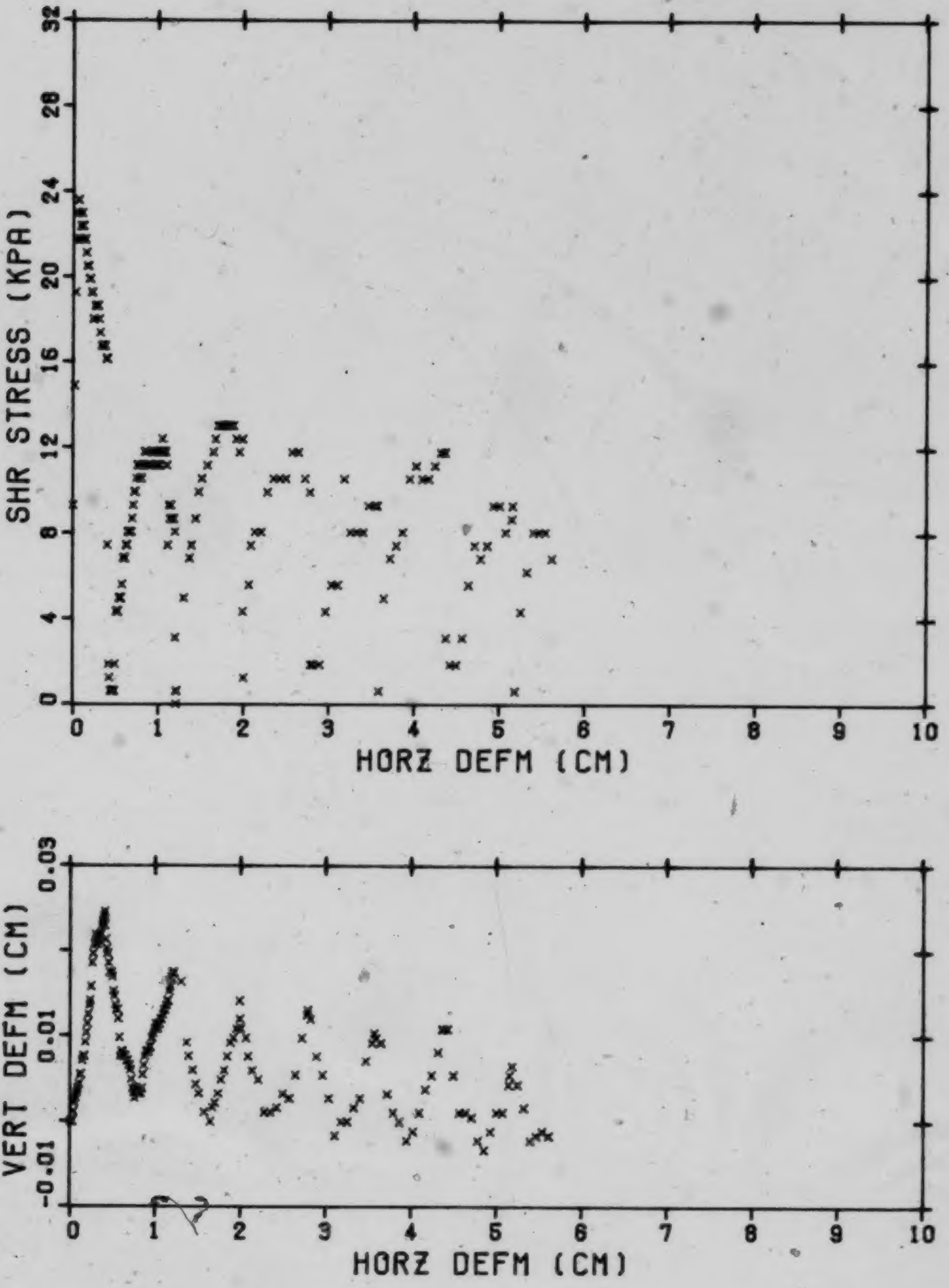
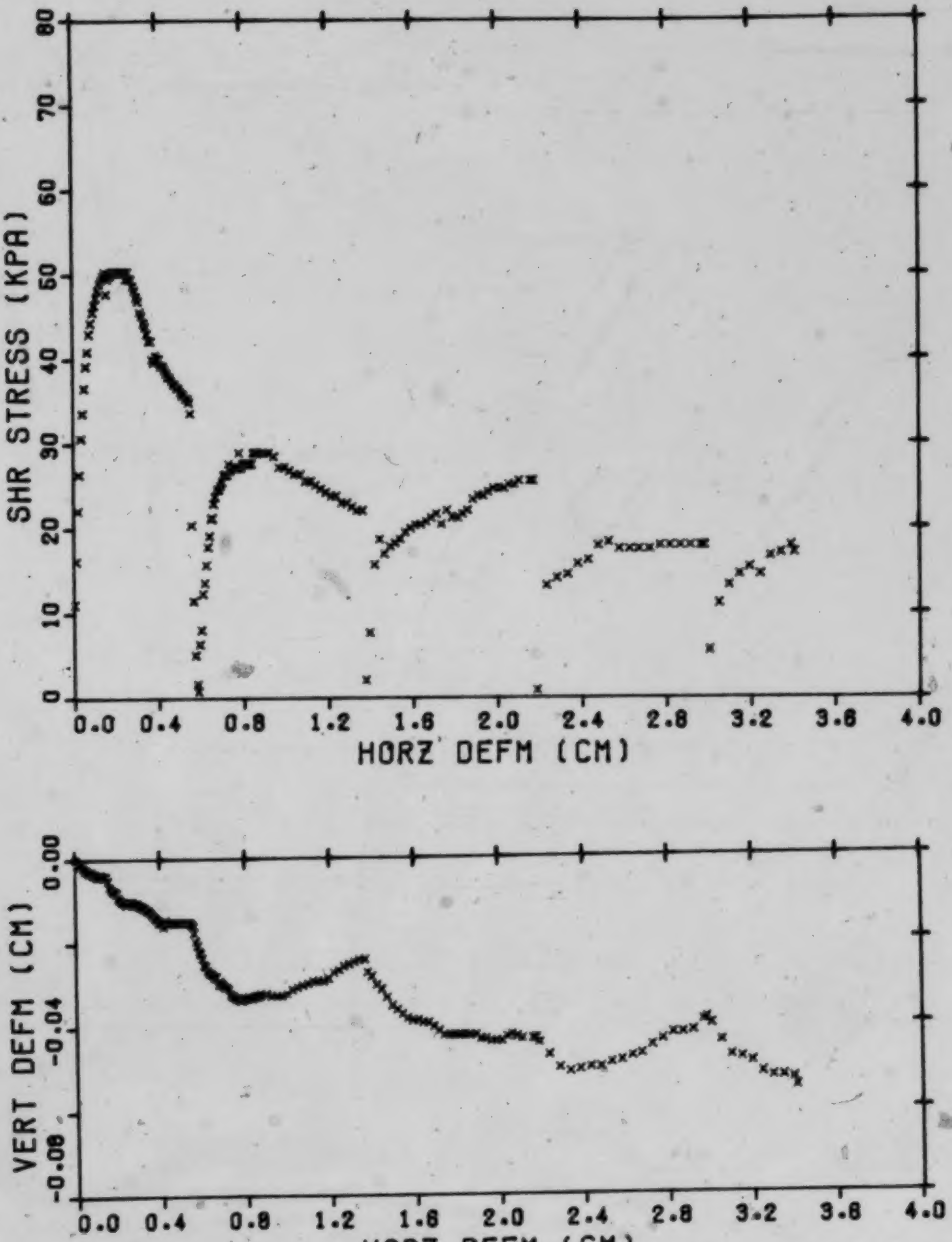
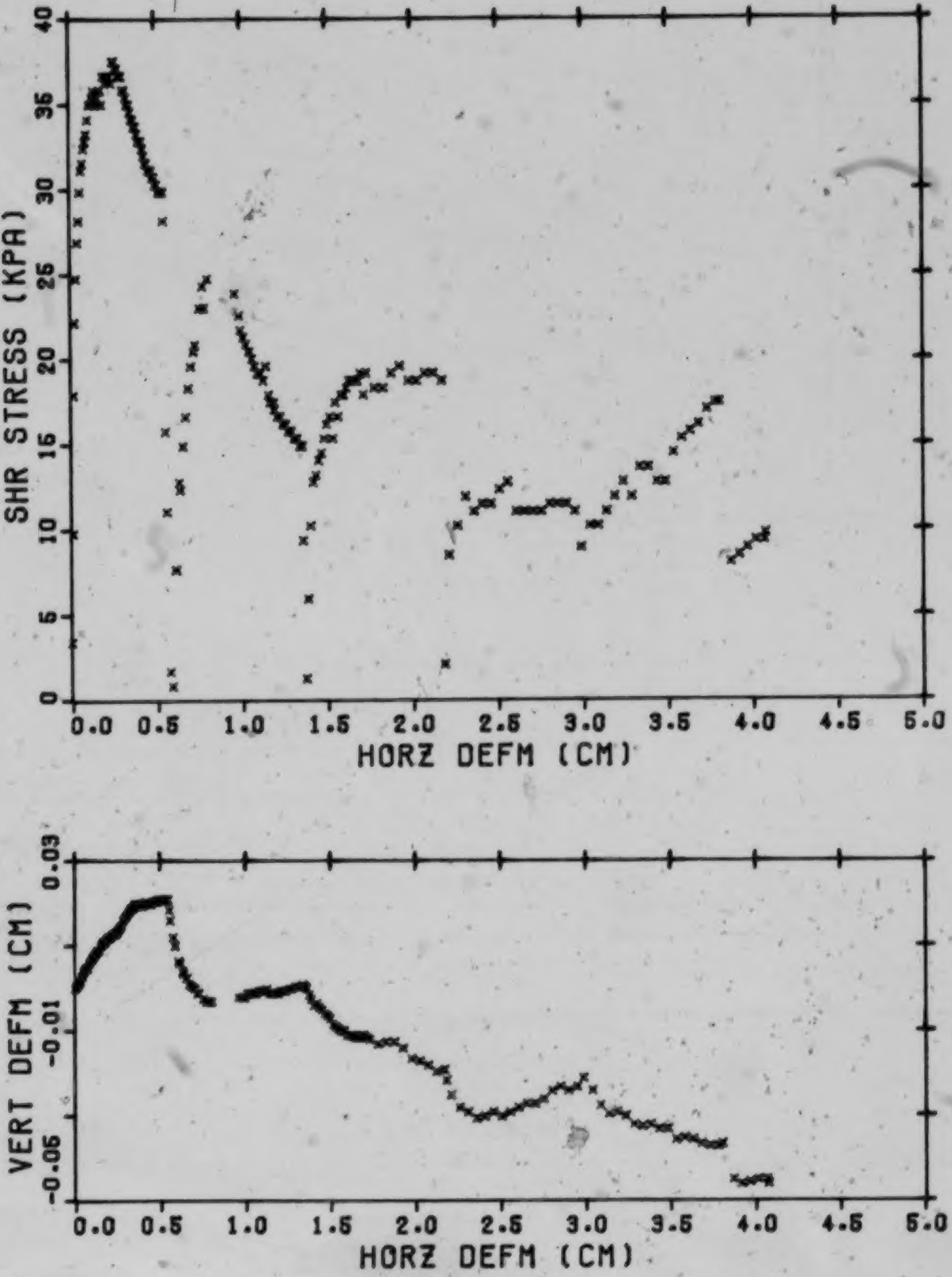


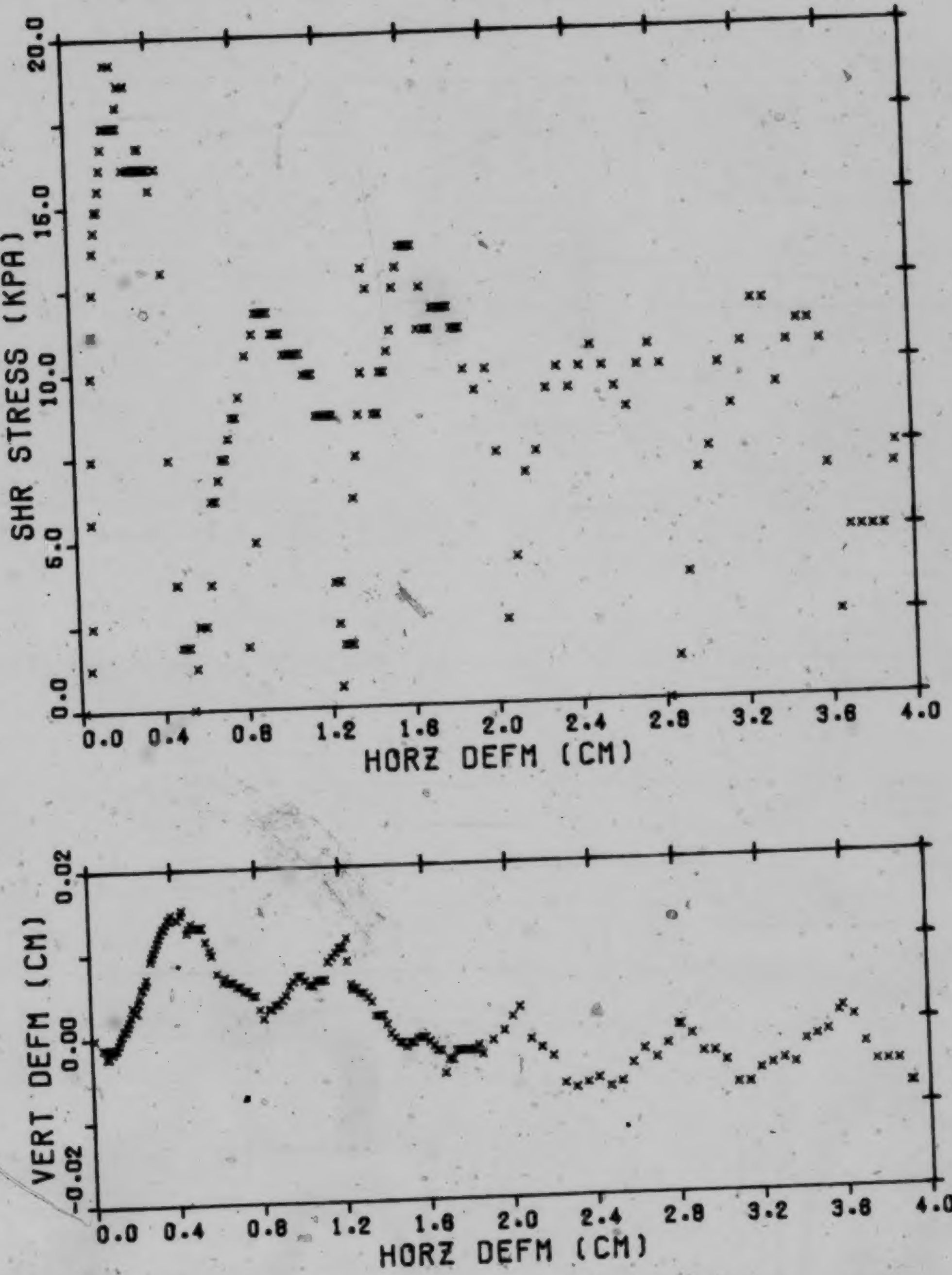
DIRECT SHEAR TEST # 4   SAMPLE T-44(B)



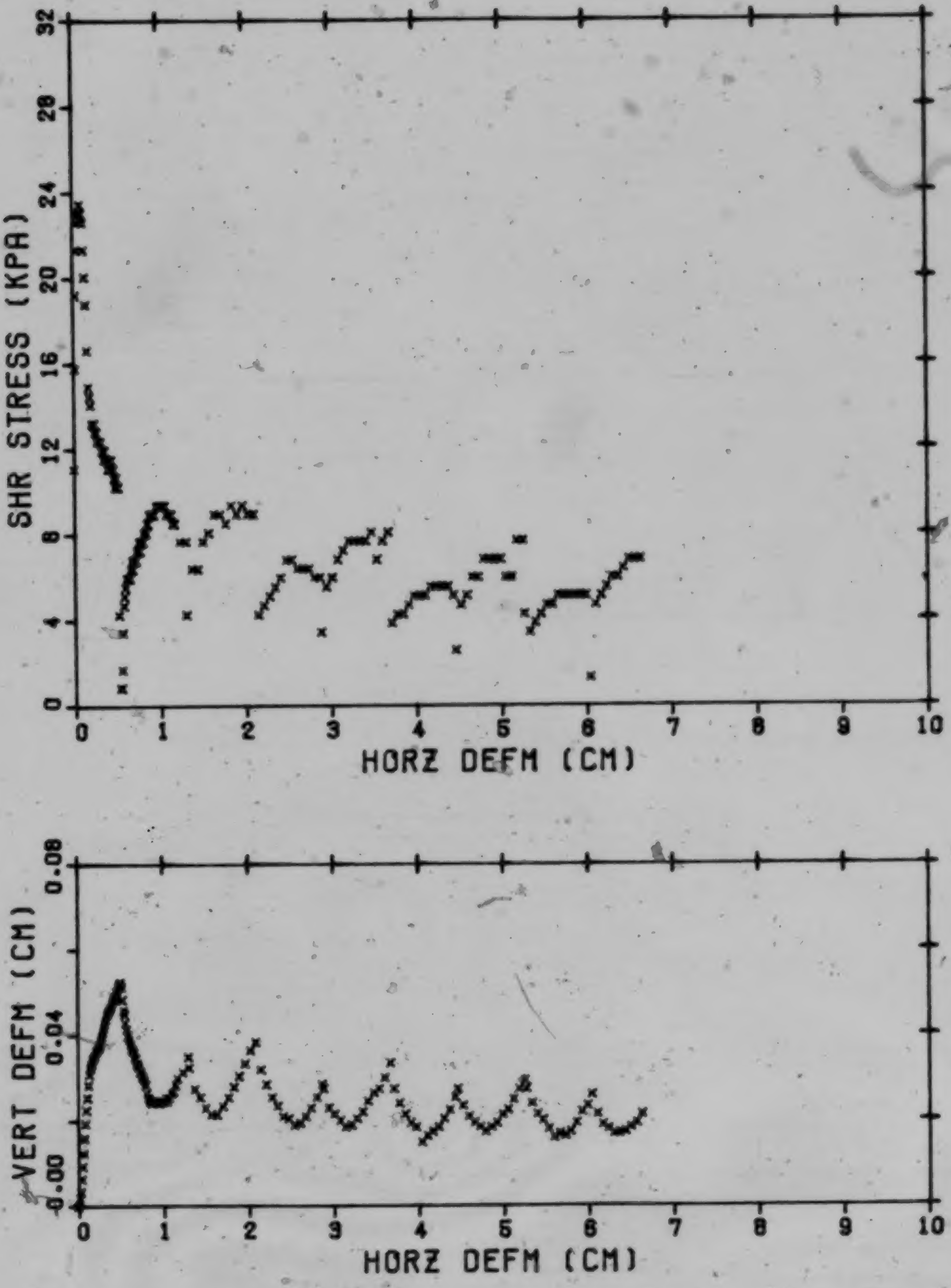
DIRECT SHEAR TEST # 5   SAMPLE T-34(A)



DIRECT SHEAR TEST # 7   SAMPLE T-40(A)



DIRECT SHEAR TEST # 8 SAMPLE T-38(A)



DIRECT SHEAR TEST #10 SAMPLE T-39(B)

APPENDIX B

Edmonton Rainfall Records

PRECIPITATION RECORDS - EDMONTON MUNICIPAL AIRPORT WEATHER STATION

(ENVIRONMENT CANADA) LAT: 53° 34'N. LONG: 113° 31'W. ELEV. 2219'

YEAR	JAN.*	FEB.*	MAR.*	APRIL+	MAY	JUNE	JULY	AUGUST	SEPT.□	OCT.▪	NOV.*	DEC.*
1967	26.2	19.8	31.0	13.5	40.1	43.7	51.3	74.4	0.8	40.9	23.1	25.9
1968	25.2	4.8	9.9	16.8	2.5	54.4	76.2	84.8	37.3	9.9	3.1	27.7
1969	23.9	19.8	8.9	28.7	33.3	24.9	83.6	114.3	80.0	25.9	17.8	17.8
1970	14.7	11.9	22.1	6.9	28.7	91.2	156.8	22.6	34.0	35.6	25.9	18.5
1971	42.7	4.1	18.5	2.8	10.9	97.5	127.0	11.9	22.4	3.6	25.2	35.3
1972	21.1	30.0	44.7	35.1	56.4	112.3	50.5	77.2	37.6	12.7	21.6	17.3
1973	11.7	12.7	5.3	39.4	37.1	147.6	61.5	101.1	47.2	31.0	34.0	27.9
1974	40.4	24.4	34.0	23.9	49.0	123.7	126.2	29.2	45.5	4.6	2.0	24.4
1975	13.2	18.3	16.3	30.0	39.9	85.9	33.5	121.2	8.4	16.1	4.4	37.5
1976	15.4	23.5	10.2	12.8	16.5	90.8	29.6	133.1	29.0	9.4	3.7	41.7
1977	21.2	6.7	12.9	19.4	138.3	18.8	141.6	74.0	37.3	0.2	12.3	22.3
NORMAL	25.2	20.1	16.8	23.4	37.3	74.7	83.3	71.6	35.8	18.5	18.5	21.3

Notes: precipitation in mm.

* greater than 90% precipitation is snowfall (on average).
+ about 10-50% is rainfall.
□ greater than 70-80% of total precipitation is rainfall.
▪ about 50-75% total precipitation is rainfall.